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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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CATENARY TROLLEY CONSTRUCTION.

By Oliver S. Lyford, Jr., M. Am. Soc. C. E.

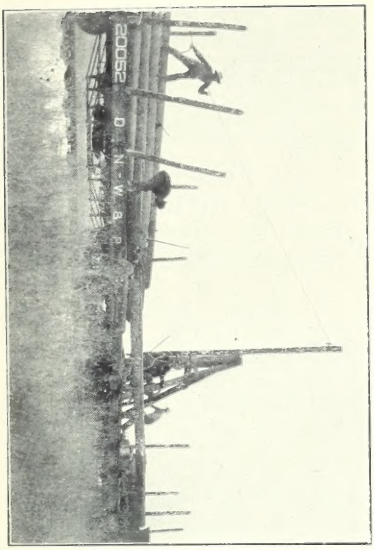
To be Presented October 7th, 1908.

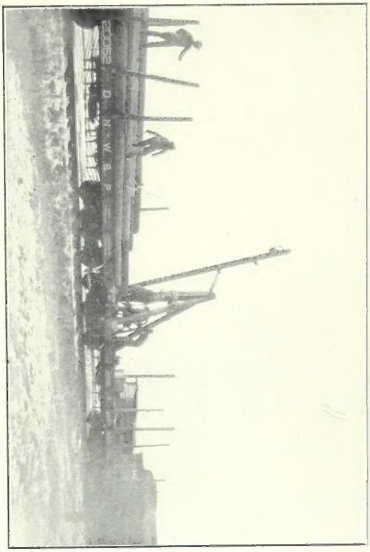
The catenary trolley work of the Denver and Interurban Railroad, in details of design and in methods of construction, exemplifies the special features involved in the problem of providing an overhead system for electric operation on roads using steam for motive power previous to and during electrification. Such roads generally have rigid regulations, and the operation of steam trains during the construction period imposes material limitations upon the work. The object of this paper is to outline some of the governing conditions relating to catenary construction for such roads, and the means adopted to meet these conditions in this most recent instance.

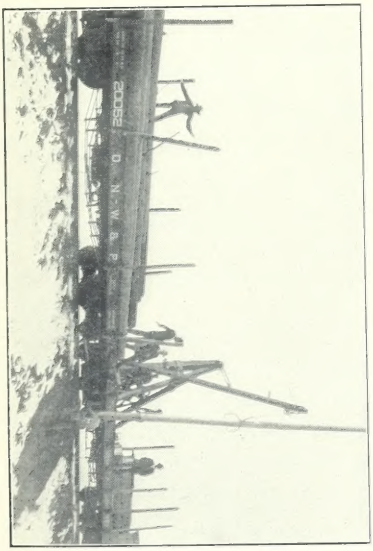
The Denver and Interurban Railroad is a part of the Colorado and Southern Railway System. This is the seventh large railroad system to inaugurate electrical operation on its lines, the other roads being the following:

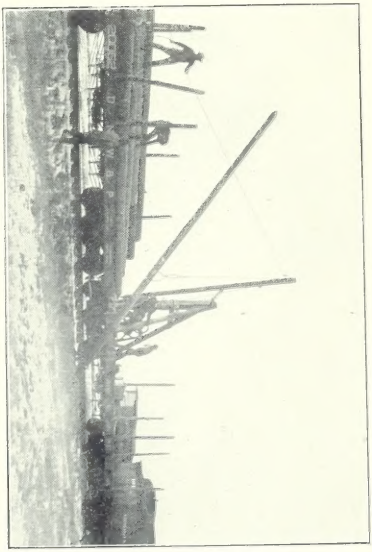
Baltimore and Ohio Railroad: 750-volt third-rail in Baltimore Tunnel;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.









DERRICK CAR FOR SETTING TROLLEY POLES ON THE DENVER AND INTERURBAN RAILROAD.

New York, New Haven and Hartford Railroad: 600-volt trolley on certain branches, and 11 000-volt single-phase catenary trolley on the New York Division and other branches;

Long Island Railroad: 600-volt third-rail on suburban lines, and 2 200-volt single-phase catenary construction on interurban line;

Pennsylvania Railroad: 600-volt third-rail on West Jersey and Seashore Division;

New York Central Railroad: 600-volt third-rail on main line, and 600-volt overhead trolley on West Shore Division and on branches;

Erie Railroad: 11 000-volt single-phase catenary trolley on Rochester Division, and 600-volt direct-current trolley on other branches.

Other large steam roads, now installing electrical equipment, are as follows:

Grand Trunk Railroad: 3 300-volt single-phase catenary trolley in Sarnia Tunnel and approaches;

Michigan Central Railroad: 650-volt third-rail in new Detroit Tunnel;

Great Northern Railroad: 6 000-volt three-phase trolley in Cascade Tunnel;

Southern Pacific Railroad: 1 200-volt direct-current trolley on Alameda Mole suburban lines at Oakland, Cal.;

Pennsylvania Railroad: New York Terminal and Tunnels—electrical system not determined.

The foregoing lists do not include any interurban trolley lines which were originally constructed as electric roads, or the few short steam lines which have been changed over to electric roads.

The Denver and Interurban Railroad is the fifth to adopt the single-phase system, and the third to use 11 000 volts on the trolley.

In the foregoing lists there are five roads using the direct-current third-rail system, two using the 600-volt direct-current trolley system, five using the single-phase system, one using the 1 200-volt direct-current trolley system and one using the three-phase alternating-current system. The last two systems have not met with general favor in America,

and the single-phase system has been selected by most of the railroad companies whose decision to electrify has been made since the commercial success of this system has been demonstrated. The principal merit in the single-phase system lies in the ability to operate at high voltage with a single overhead conductor, and, therefore, to transmit power long distances with a minimum number of sub-stations and a minimum quantity of copper. These arguments apply particularly to trunk-line operations, and this system, therefore, appeals especially to the managers and engineers of large railroads. On the Denver and Interurban Railroad, the extent of curvature, the number of cuts and fills, parallel tracks, sidings, and other local features have introduced into this undertaking many of the principal construction problems which are likely to develop in a single-track road with moderately heavy traffic. Of the total length of track of this line, 43% is on curves.

The lines electrified are shown on the map, Fig. 1, and consist of a main line from the city limits of Denver to Louisville Junction, and two branches thence to Boulder. There is also a spur from the Marshall Branch to Eldorado Springs. The portion outside the terminal cities is equipped with the single-phase system. Within the limits of each terminal, operation is by the 600-volt direct-current trolley system. The cars, therefore, are equipped with electrical apparatus suitable for operation by either system. There are 44 miles of single-phase construction. In Boulder, the railroad company has provided 1⅓ miles of 600-volt direct-current construction. In Denver, the cars are operated on the city streets, over the lines of the Denver Tramways Company.

The principal features involved in the design and construction of such overhead work are outlined in the following notes:

Sufficiency.—The first cost of electrification work for railroads is high, but the cost of electrical operation is low, as compared with steam-locomotive operation, provided the equipment is properly designed, erected and maintained. High-voltage catenary work cannot receive the close daily inspection of the track-walker, and, because this is so, there is the greatest necessity for thoroughness and permanence in the construction. Intelligence and experience, therefore, are at a premium in this field of engineering and operation.

Rapid Construction Work.—The details of design and methods of

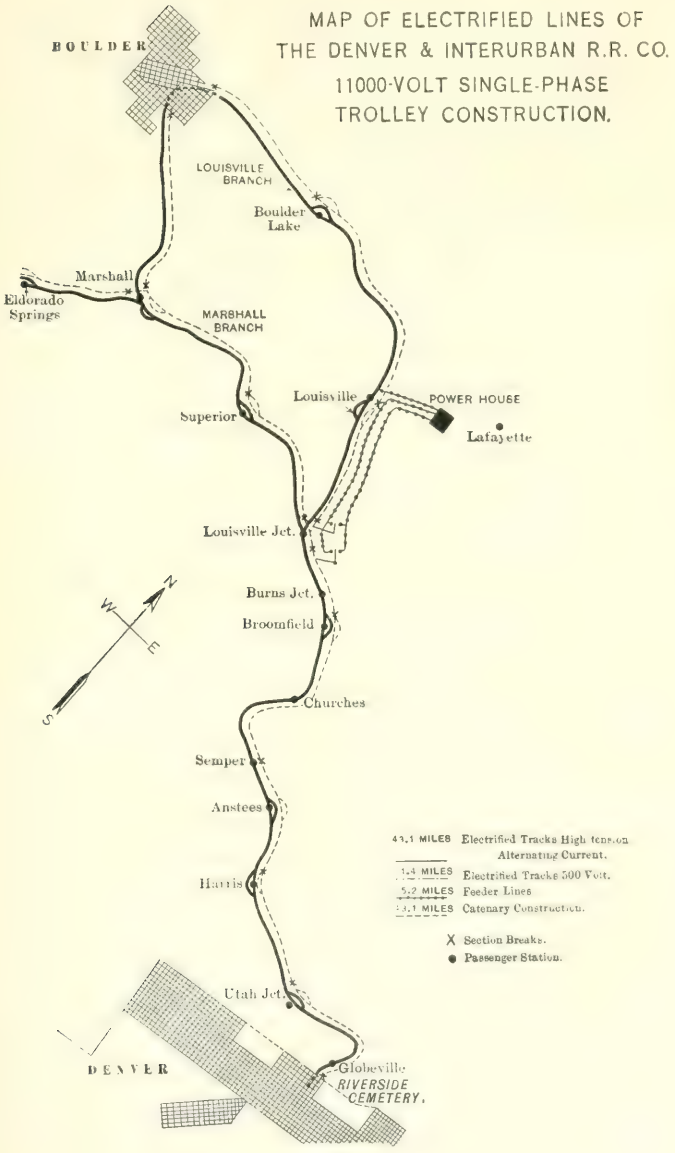

MAP OF ELECTRIFIED LINES OF
THE DENVER & INTERURBAN R.R. CO.
11000-VOLT SINGLE-PHASE
TROLLEY CONSTRUCTION.
BOULDER
LOUISVILLE BRANCH
Boulder Lake
Marshall
Eldorado Springs
MARSHALL BRANCH
Superior
Louisville
POWER HOUSE
Lafayette
Louisville Jct.
Burns Jct.
Broomfield
Churches
Semper
Anstees
Harris
Utah Jct.
DENVER
Globeville
RIVERSIDE CEMETERY.
N
W
E
S
43.1 MILES Electrified Tracks High tension Alternating Current.
1.4 MILES Electrified Tracks 500 Volt.
5.2 MILES Feeder Lines
3.1 MILES Catenary Construction.
X Section Breaks.
● Passenger Station.

FIG. 1.

construction must be worked out with special reference to rapid erecting operations. Time spent in the drafting-room in developing details which go together quickly means much time and expense saved in the field, where each construction operation is repeated hundreds of times. Much ingenuity can also be exercised in planning the field work. The accompanying photographs illustrate the quick methods of construction used in the Denver and Interurban work.

Work Train.—Much depends upon the proper equipment of the work train. For pole-setting, a derrick car is provided (Plate LXVI), and, for overhead work, box cars are equipped with platforms at the proper height for convenient operation (Fig. 1, Plate LXVII). Where overhead obstructions exist, these platforms must be collapsible. The interiors of the box cars are arranged with storage bins, workbenches, tools, etc. Where construction is interfered with by the operation of schedule trains, the preparation work proceeds while the work train is on the siding, and everything is arranged for progress with the utmost despatch as soon as the main line is available.

Poles.—Poles are set at a distance of 8 ft. 6 in. from the center of the track to the inner side of the pole. This is sufficiently close to the track so that pole setting direct from the cars can be done to great advantage. The figures on Plate LXVI illustrate such work. A record day's work with such an equipment, without interruption, is 117 poles set, including digging holes and tamping. This was done with 26 men. In one instance, 13 poles were placed in their holes in 15 min.

Bracket-Setting.—The effectiveness of the work train for bracket-setting depends somewhat on the extent to which the line may be kept free from regular trains. Where interference is infrequent, the work train can be used to advantage in the manner indicated in Figs. 2 and 3, Plate LXVII. The locomotive places the cars opposite the poles, and bracket erection proceeds at four or five poles simultaneously. As soon as the brackets are in place, the locomotive couples up the cars and moves on. The brackets, of the type shown in Fig. 3, Plate LXVII, can be erected complete and adjusted to the right height at the rate of 150 per day, with 18 men and a work train of 5 cars. Fig. 3, Plate LXVII, shows the support placed on the top of the car for setting the bracket at the proper height, allowance being made in this setting for the deflection which will occur with the load of the catenary structure.

Placing Wires.—For placing wires the derrick car is arranged to

FIG. 1.—WORK TRAIN FOR ERECTING BRACKETS AND CATENARY.

FIG. 2.—WORK TRAIN DEPLOYED FOR ERECTING BRACKETS.

FIG. 3.—ERECTING BRACKETS FROM WORK TRAIN.

FIG. 4.—APPARATUS FOR STRINGING WIRES.

carry the reels and pay the wire out over elevated rollers (Fig. 4, Plate LXVII). The best day's run on messenger wire was 7 miles strung and tied in. This took 17 workmen and two train crews. The best day's run on stringing and splicing trolley wire was also 7 miles, with 12 workmen and one train crew. The best day's record of clipping up messenger and trolley wire was 3½ miles with 13 workmen and one train crew. A working day consisted of 9 hours, with transportation from headquarters each day on the company's time, leaving about 7 working hours. The ground-wires and feeder-wires are placed before the brackets are erected, and the wire is paid out over the end of the boom of the derrick.

Adjusting the Sag.—The intention, in the design of catenary construction of this type, is that the trolley wire shall remain as nearly flat as possible, and the tension in the messenger wire and trolley wire must be determined with considerable definiteness between the minimum, which will prevent excessive deflection, and the maximum, which must be within safe limits for the material.

In the Denver and Interurban work the sag table used was that given in Fig. 2. To verify this table a special dynamometer was provided, consisting of a calibrated car-spring. The setting of the tension in the messenger and trolley wires by using this dynamometer on the top of the work train is illustrated in Fig. 1, Plate LXVIII. The use of this dynamometer disclosed a point of interest in connection with the erection of long lengths of wire and cable. For an average sag of the messenger cable, which, according to the sag table, should result in a tension of 2 300 lb., the dynamometer showed an initial tension of 6 000 lb. This initial strain decreased to 4 000 lb. before quitting, and, after expansion and contraction over night, came down to the figure in the sag table. This excessive initial strain occurred on a portion of the line which is full of curves, and was due to the lack of equalization between the different spans of a 1-mile section. On tangents, the initial strain was between 3 500 and 4 000 lb. This is a point to be borne in mind in calculating the proper elastic limit for such wires and cables.

Pole Collars.—Experience with semaphores and other heavy structures on wooden poles has shown that such poles deteriorate most rapidly at the ground line and at points where bolts pass through the pole. With this experience in mind, the Denver and Interurban

brackets have been attached to the poles with pole collars (Fig. 3, Plate LXVII), and no bolts pass through the poles.

Treatment of Pole Butts.—To increase the life at the ground line, the poles have been treated with carbolineum.

Ground-Wire.—In the Denver and Interurban work, a ground-wire, consisting of $\frac{7}{16}$-in. stranded galvanized-steel cable, has been strung over the pole tops for the entire length of the line. This wire has a number of functions:

It affords means for grounding the brackets and cross-spans so that working upon these parts is safe;

The ground connection of the running rails can be made at every fifth pole instead of at every pole, as would be necessary without this cable;

It affords a third circuit in parallel with the two running rails for the return current;

Being at ground potential and at an elevation above the trolley wire, it affords effective lightning protection;

It is a cheaper construction than the grounding of each bracket to the running rail and the addition of lightning arresters.

Track Connection of Ground-Wire.—Cross-bonding of the two running rails is necessary at frequent intervals. By placing these cross-bonds opposite every fifth pole it is possible to use the cross-bond as a means of connecting the ground-wire to the track, as shown in Fig. 6. A track connection of this kind is subject to abuse by the section men, and also from the vibration due to trains. A rather heavy ground-rod is necessary for protection from the section hand, and flexible connection to the rail is necessary because of the action of the trains. The combination of these requirements is best provided for in the manner illustrated. The conductor in this case is a $\frac{5}{16}$ by $1\frac{3}{4}$-in. iron strap. The flexible connection to the rail is obtained by attaching this strap to the center of the cross-bond. This gives a connection to each rail. The cross-bond is subject to the abuses of track repairers, but to a minimum extent. The figure illustrates the track connection for cross-span construction, but the same arrangement is used for bracket poles.

Grounding the Running Rails.—In order to avoid any considerable

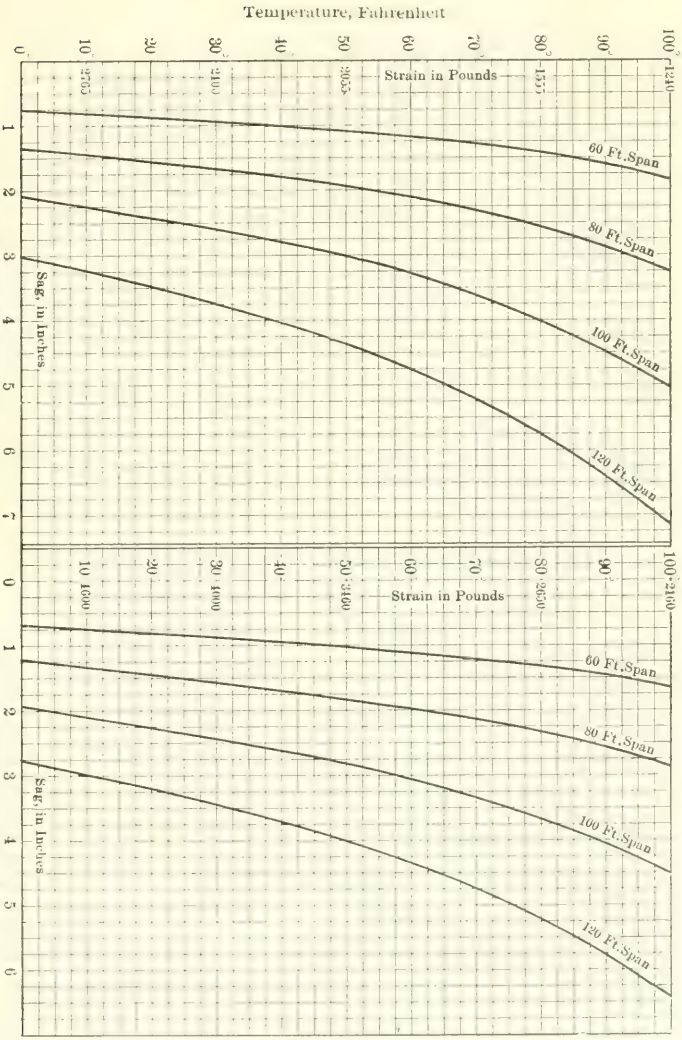

Temperature, Fahrenheit
Strain in Pounds
60 Ft. Span
80 Ft. Span
100 Ft. Span
120 Ft. Span
Sag, in Inches
CURVES SHOWING ERECTING SAGS
IN 5/16" MESSENGER CABLE
(BEFORE HANGERS ARE ATTACHED)
MODULUS OF ELASTICITY 23000000
Strain in Pounds
60 Ft. Span
80 Ft. Span
100 Ft. Span
120 Ft. Span
Sag, in Inches
CURVES SHOWING ERECTING SAGS
IN ½ PHONO-ELECT. TROLLEY
(BEFORE THE TROLLEY WIRE IS PLACED IN HANGERS)
MODULUS OF ELASTICITY 13500000

Fig. 2.

potential between the running rails and the ground, it is necessary to provide ground-plates at frequent intervals, with adequate connections between these and the running rails.

Iron and Steel Work.—Most of the iron and steel parts used in catenary construction are small in cross-section. For long life, a preservative coating is necessary. Proper inspection, cleaning, and painting are accomplished with difficulty, and, although such care is essential, there is considerable probability of neglect. A short-lived coating, therefore, is not sufficient. Galvanizing seems to be the only suitable method of protection thus far developed. The combined action of the elements and locomotive gases on galvanized work has been watched carefully. The locomotive exhaust forms a hard deposit on wires placed directly above the center of the track, and this deposit seems to act as a preservative coating over the galvanized surface. In this work there is special necessity for ample thickness of ungalvanized sections, because of the difficulty involved in adequate inspection and painting.

Trolley Wire.—It is essential that the trolley wire shall at all times be under sufficient tension to avoid appreciable sag between the 10-ft. supports. With catenary construction, of the type herein referred to, a tension of at least 2 000 lb. is necessary in order that the pantagraph shoe may ride smoothly under the trolley wire. With lower tension there will be chattering of the pantagraph, and abnormal wear of the pantagraph shoe and trolley wire. If the minimum tension at 100° fahr. is 2 000 lb., the theoretical strain at 0° fahr. will be approximately 5 000 lb., as shown in the curve sheet of sags, Fig. 2. The actual strain will be some indeterminable figure less than this, depending on the general elasticity of the system. The average tensile strength of a 4/0 hard-drawn copper wire when new is 50 000 lb. per sq. in., or 8 310 lb. for a 4/0 wire. The elastic limit is 35 000 lb. per sq. in., or 5 817 lb. for a 4/0 wire. As the copper wire is reduced by wear, the tensile strength is reduced more rapidly than the physical section, because the center is not as strong as the skin. "Phono-electric" wire has physical characteristics which are much better suited to this service, as follows:

Tensile strength: 68 780 lb. per sq. in., or 11 330 lb. for a 4/0 wire:

FIG. 1.—PUTTING THE STRAIN ON TROLLEY WIRES.

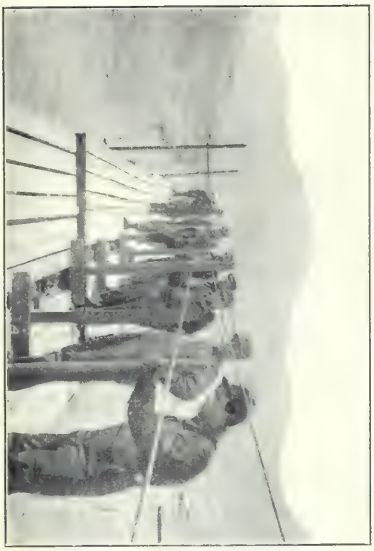


FIG. 2.—GAUGING THE CATENARY STRUCTURE.

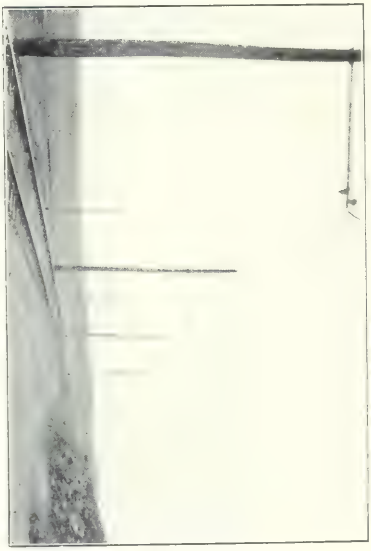


FIG. 3.—COMPLETED CATENARY WORK ON A CURVE.

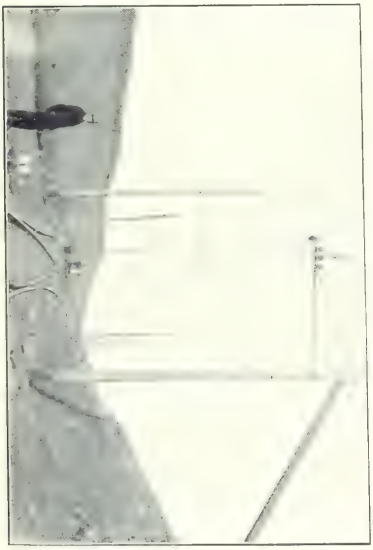


FIG. 4.—TURNOUT.

Elastic limit: about 58 000 lb. per sq. in., or 9 640 lb. for a 4/0 wire;

The material is homogeneous throughout;

The material is considerably stiffer than hard-drawn copper;

The wire may be worn to the limit permissible by the hanger fastenings before the strain at minimum temperature will equal the elastic limit of the reduced section.

The conductivity of this material is about 45% of that of hard-drawn copper, but, in alternating-current railway work, this is of less importance than in direct-current work, for several reasons. The track resistance is much higher than with direct current, because of the self-inductive effect of the rails, which are of magnetic material. There is, also, an additional drop in the circuit, due to the induction between the trolley wire and the rails.

Taking all these factors into account, the increase in the combined drop, resulting from the use of this material with low conductivity, is only 38 per cent. Furthermore, as the circuit has to be calculated for the maximum load, and the daily average is a small proportion of this, the increase in the average energy loss is inconsiderable.

In view of the cost of labor and supplies in renewing the trolley wire, and the interference with operation while this work is going on, the advantage of long life is apparent. Therefore, the physical advantages are largely in favor of the Phono-electric wire, and the electrical disadvantages which offset these are not serious. This material is used in the Denver and Interurban work.

Splicing Sleeves.—Mechanical splicing sleeves, in which the trolley is bent on short radius, do not preserve the full strength of the trolley wire, as the outer fibers on the bends are strained beyond their elastic limit. For this catenary work, where the higher strains are necessary, as above stated, it is essential that the splicing sleeve, including the ends of the trolley wire connected thereto, shall have the same strength as the main body of the wire. To this end, the wire must enter the splicing sleeve with practically no bends. The best sleeve thus far developed is one of the style indicated in Fig. 3, in which the joint is made with solder.

Trolley Hangers.—The trolley wire is supported from the messenger wire at intervals of 10 ft. The supporting device, or "hanger,"

varies in length according to the sag of the messenger cable. As many thousands of these hangers are used, it is important that the design be suited to the special conditions of erection and operation. The Denver and Interurban hanger, illustrated in Fig. 4, embodies the following features:

The trolley clamp and hanger rod can be assembled in the work car before erection;

The operations necessary when erecting are such that they can be performed with the utmost speed;

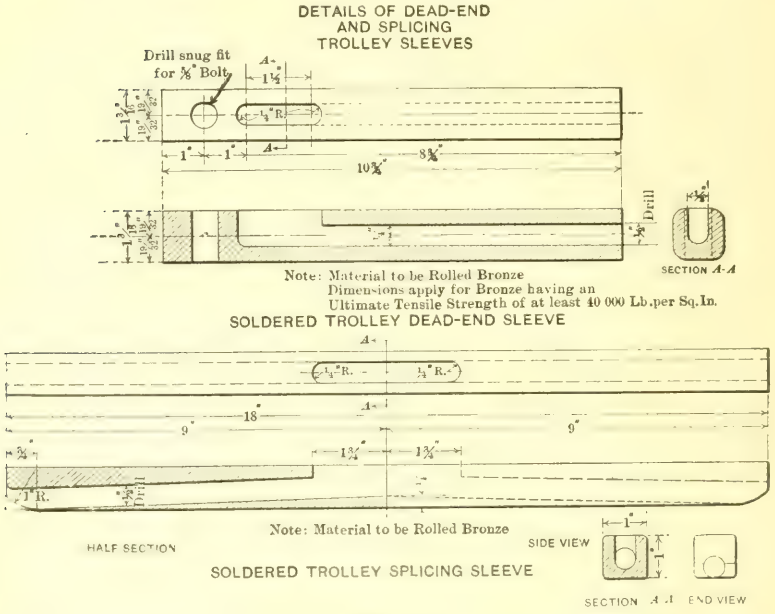


FIG. 3.

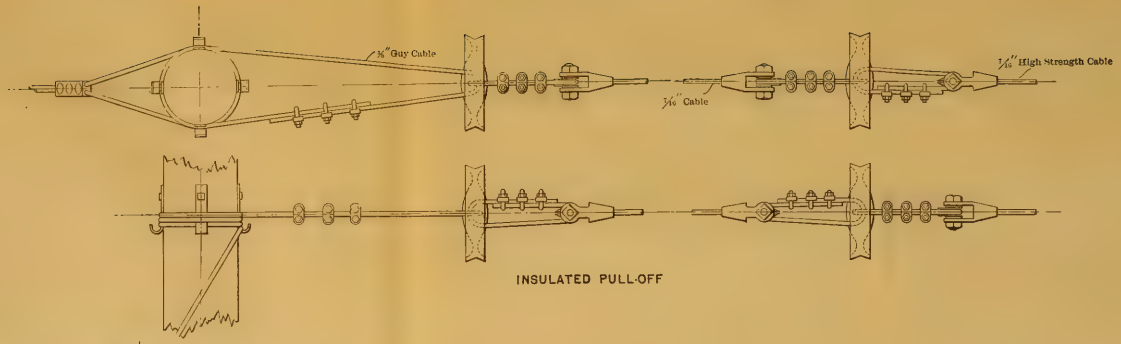
The hanger can be easily loosened, when necessary to pull the slack out of the trolley wire;

It has ½ in. adjustment in length;

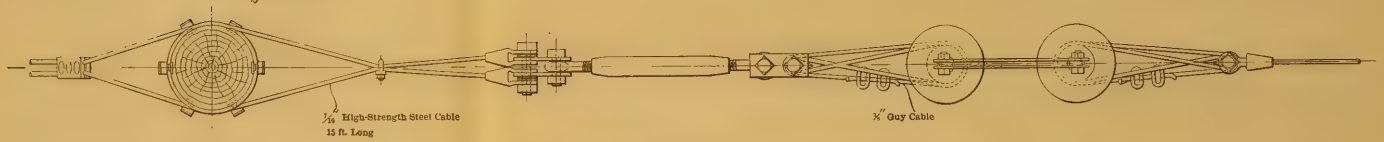
All nuts move freely, but lock in position;

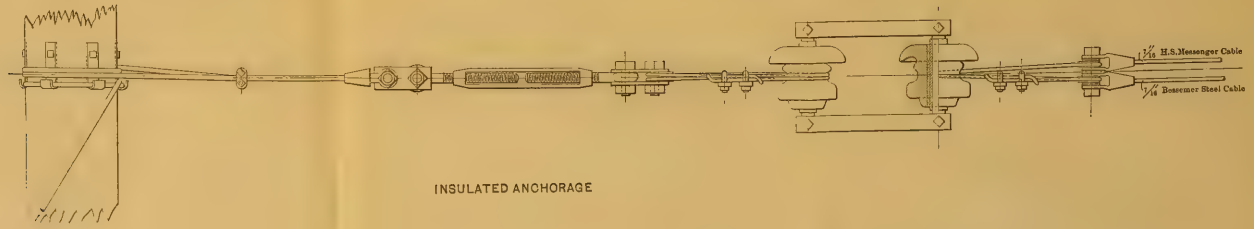
It is strong, but light.

Insulation.—An insulation failure on a high-tension electric system means the loss of power over the entire section in which the trouble occurs. This may mean that a number of trains will be brought to a



INSULATED PULL-OFF





INSULATED ANCHORAGE

stand-still, and, unless the trouble is quickly located, a considerable blockade will result. This makes it necessary that high-voltage trolley systems be put up so as to minimize line troubles. This condition has led to exhaustive investigation of different types of insulators and different methods of construction. Porcelain is the best material thus

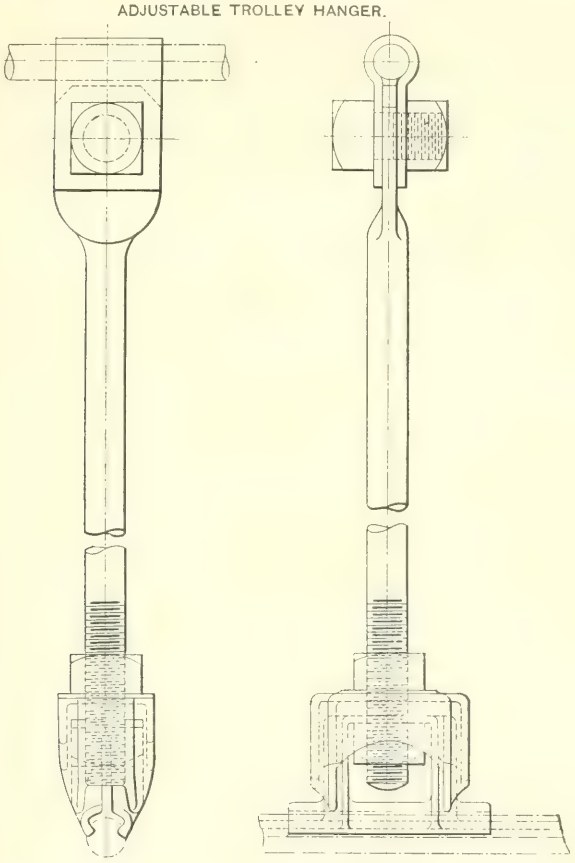


FIG. 4.

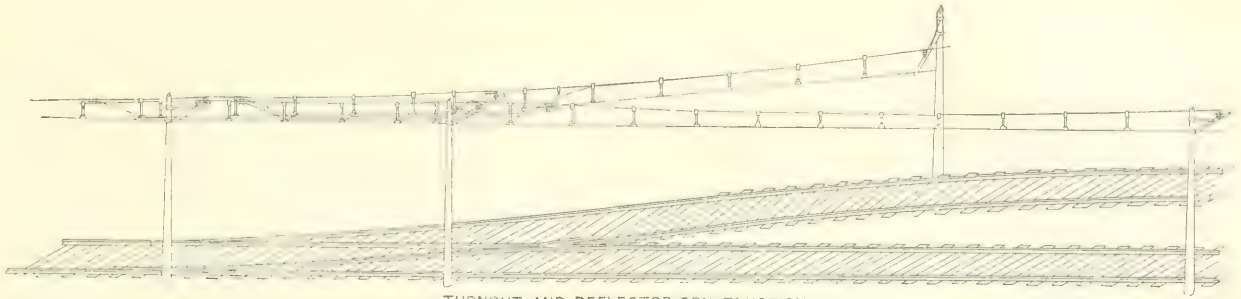
far found for insulating purposes, and, with sufficient factors of safety against mechanical strain and electrical tension, catenary structures insulated with porcelain have been found very reliable. The insulator used in straight-line work in the present case is illustrated in Fig. 1, Plate LXX. This is a triple-petticoated insulator designed for regular

operation at 22 000 volts. Each insulator was tested at the factory with 70 000 volts when dry and with 43 000 volts when subjected to an artificial rain. The same insulator is used in the "steady strain" described below. For pull-offs, a disc-type insulator is used, as illustrated in the upper part of Plate LXIX. This insulator will stand a combined mechanical strain of 6 500 lb., and electric tension of 30 000 volts. For dead-ending work, spool-type insulators are used, as illustrated in the lower part of Plate LXIX. These insulators, when properly harnessed, as shown in Plate LXIX, can be subjected to a combined test of 14 000 lb. compression and 55 000 volts electric potential. All strain insulators are used two in series so as to double the electrical factor of safety; and the harness is interlaced, so that, should one insulator break, the structure will not fall to the ground, but the remaining insulator will be adequate, electrically, to permit the continuation of operation until repairs can be effected.

Steady Strain.—To keep the trolley wire in a position vertically below the messenger wire on curves, it is necessary to use a special device known as a "steady strain." The construction must be such that in no case will it interfere with the pantagraph trolley. The steady strain must be insulated from the supporting bracket. The insulator and the means of fastening it to the bracket are necessarily heavy, and, should they become disconnected from the bracket, would sag to a position where they would be below the plane of the pantagraph. To provide against this emergency the steady strain used on the Denver and Interurban road is of a new design, and is arranged so that in case of breakage the insulator and the parts attached thereto are automatically released from the member attached to the trolley wire and fall to the ground, thus preventing interference with the pantagraph. This device (Fig. 1, Plate LXX) is inexpensive and effective.

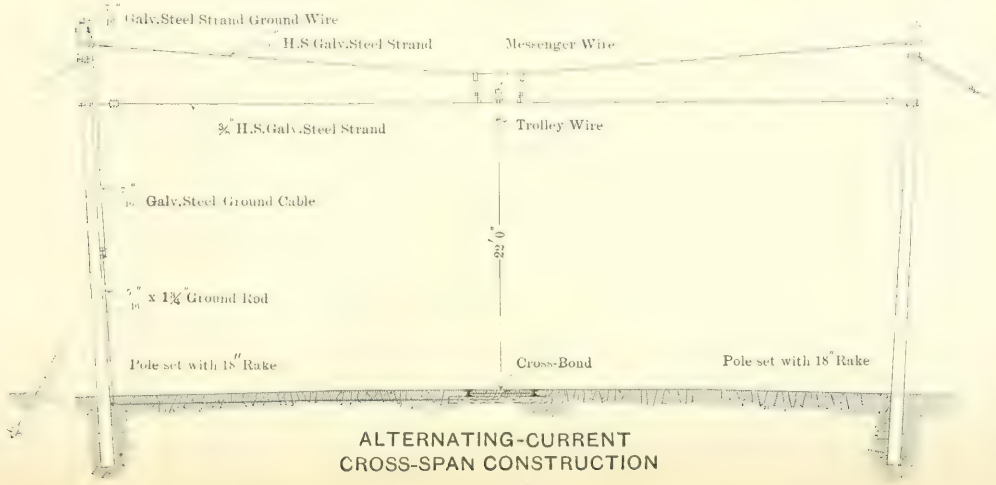
Deflector.—To avoid the possibility of the pantagraph becoming caught in the diverging wires of a turn-out, it is customary to provide an arrangement called a "deflector," consisting of intermediate wires suitably mounted between the two trolley wires. The device used in the case under discussion is illustrated in Fig. 5, and also in Fig. 4, Plate LXVIII, and Fig. 2, Plate LXX.

Cross-Spans.—The cross-span construction for 11 000-volt work of the Denver and Interurban Railroad is illustrated in Fig. 6. The entire structure is grounded for safety.



TURNOUT AND DEFLECTOR CONSTRUCTION.

FIG. 5.



ALTERNATING-CURRENT
CROSS-SPAN CONSTRUCTION

FIG. 6.

Offset of Trolley Wire.—In the Denver and Interurban work the pole setting on curves has been such as will permit of keeping the trolley wire within 3½ in. of a line directly above the center of the track. The poles were spaced according to the figures in Table 1.

TABLE 1.—POLE SPACING.

Degree of curvature of track.	Pole spacing, in feet.	Divergence of trolley wire from track, in inches.
Tangent.	120	0.
1	120	1.87
2	110	3.37
3	90	3.06
4	80	3.37
5	70	3.06
6	60	2.87
7	50	2.34
8	50	2.64
9	50	2.94
10	50	3.24

Many roads have used greater offsets than shown in Table 1, in order to save poles, but it should be borne in mind that the use of an overhead conductor in a permanent position above the tracks has a material effect upon existing practice in maintaining roadbeds. Hitherto, the tracks have been constantly shifted by the section men in the correction of the alignment and in ballasting, and there have been no close limitations. With catenary work above the track, the section man is forced to work within close limits. For instance, 1 in. superelevation of one rail means 4½ in. deflection of the pantagraph. If this is a bump in the track and causes the car to sway, the divergence may be considerably more. If this happens on a curve, the deflection of the pantagraph and the offset of the wire act together. Obviously, it will be some time before section men are trained to the necessity of close work, and they should be given as much latitude as possible. This means that there should be as little offset as practicable in the original design. Furthermore, the use of small deflections means a minimum of side strain on the structure, and, therefore, a minimum likelihood of displacement of insulators, steady strains, etc. With wood poles at present prices, adherence to this practice of using small offsets does not materially affect the cost.

Advantage of Light Supporting Structure.—The use of wooden poles for catenary construction is in many respects preferable to the

FIG. 1.—LINE INSULATOR AND STEADY STRAIN.

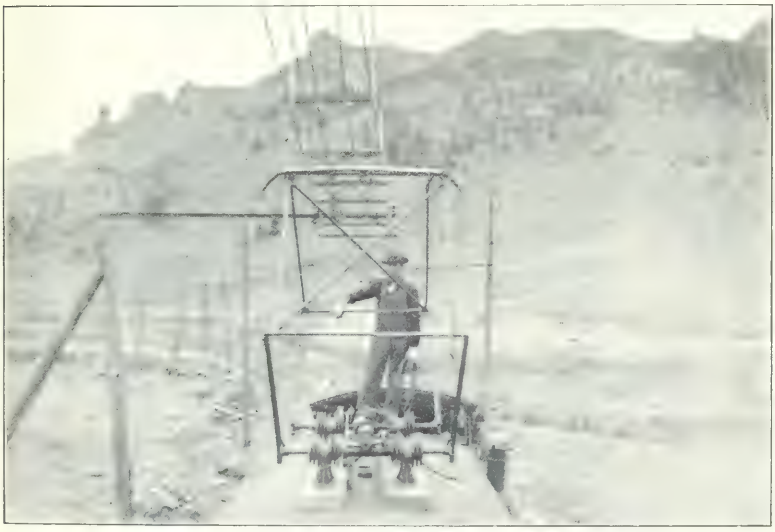


FIG. 2.—DEFLECTOR.

use of heavy steel bridges. The principal advantage is in the ease of repair. It is unfortunate for catenary work that the supply of wooden poles is rapidly diminishing. An equally light, but strong, substitute is desirable. The question naturally arises as to what will happen to the catenary structure, if the poles are struck by a derailed train. One answer to this question was given in the case of similar catenary work for the Erie Railroad. A derailment of the work train occurred during the construction period, and the caboose was driven into one of the electric poles. The pole was cut off at the ground line, the insulator torn loose, and the pole thrown to one side entirely free from the catenary work. The tie-wire, by which the messenger cable is attached to the insulator, acted as a safety connection in such a case, as, by breaking, it released the catenary structure from the bracket and pole. With the tensions used in the ordinary catenary structure of the Denver and Interurban, two adjoining poles may be destroyed and the trolley wire will only sag to 17½ ft. above the track in the maximum case.

Sectionalizing the Trolley Wire.—For convenience in locating faults which may develop in the catenary construction, it is advisable to divide the conductor into sections 4 or 5 miles in length. These sections are normally connected so as to make a continuous circuit, the connections being through switches which can be opened for test.

Clearance of Overhead Obstructions.—The proper height for the trolley wire above the running rail is 22 ft. This means a standard height of 24 ft. for the clearances of through and under bridges, and still greater clearances for other overhead obstructions wherever physical conditions permit. In a large majority of cases, new construction can be designed for 24 ft. clearance without much difficulty or material addition to cost. It is recommended that railroads adopt this as the standard height for clearances in the future, except for tunnels, municipal bridges, etc., where other conditions govern. For clearances of less than 24 ft. special constructions of the catenary work have to be provided, which are more or less objectionable.

Hard Spots in Catenary Structure.—For high-speed operation, it is essential for satisfactory operation of the pantagraph that the maximum flexibility of the overhead structure be maintained at all points. For this purpose, it is important that heavy weights and hard spots be kept out of the system wherever possible.

Bonding.—Experience with the numerous types of bonds which have been placed on the market indicates that wherever possible the bonds should be placed under the splice-bar. Bonding the tracks of a road which is in constant use is a difficult and slow operation with bonds of this type, but the advantage to be gained in the way of permanence and security warrants the trouble.

Repair Work.—In view of the fact that line trouble usually means loss of power, the work train cannot be taken to the fault by an electric locomotive; therefore an independent unit should be provided, that is, either a steam locomotive or a gasoline car, preferably the latter, because the stand-by losses on such a car are small, and it can be quickly put into service.

Power Supply.—The power for operating the cars of the Denver and Interurban Railroad is furnished by the Northern Colorado Power Company from its power-station near Louisville. For this purpose, two 1 000-kw. single-phase turbo-generators are provided. Two feeders connect this power-station with the center of distribution at Louisville Junction. The switch connections at Louisville Junction are such that both feeders may serve all three legs of the trolley simultaneously, or one feeder may serve the main line and the other feeder serve the two branches (Fig. 1). Single-phase generators necessarily cost considerably more than generators designed to deliver the same output into a three-phase system, and in some cases it is preferable to generate three-phase current and serve separate sections of the trolley line from each of the three phases. As a general rule, however, where new generating equipment is provided for single-phase operation exclusively, the advantages are in favor of the single-phase generation, even at the increased cost.

Telephone and Telegraph Interference.—Single-phase current has an inductive effect on parallel telephone and telegraph circuits, causing interference with the operation of such circuits unless special protective means are used. Simple devices have been contrived for overcoming this interference, so that there is now but little difficulty with the operation of such parallel circuits. In most of the single-phase roads there is a telephone system on the same poles as the trolley line, and, with proper transpositions and suitable means for removing the induced charge from the telephone line, it operates very satisfactorily and without shock to the operators.

Cost.—The cost of this catenary construction varies between rather wide limits, depending on the amount of electric train service to be operated, the physical conditions as to curves, grades, obstructions, and pole foundations, and the amount of traffic on the line during electrification. These conditions affect the cost of both material and labor. The labor operations mentioned in this paper are those performed under most favorable conditions. It has been found that, because of conditions beyond the control of the designer or the construction crew, the amounts of work per day may be cut to one-third, or even as low as one-seventh, of the figures given herein. In general, it may be said that, under favorable conditions, 11 000-volt catenary construction on wooden poles, with conductors sufficient for a half-hourly operation of two-car trains, and including track bonding, costs from $3 500 to $5 000 per mile of single track electrified.

Vol. XXXIV. AUGUST, 1908. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE SINKING OF THE PIERS FOR THE GRAND TRUNK PACIFIC BRIDGE AT FORT WILLIAM, ONTARIO, CANADA.

By H. L. Wiley, Jun. Am. Soc. C. E.

To be Presented September 16th, 1908.

This paper outlines the construction methods used in sinking the piers for the Grand Trunk Pacific Railway Bridge, crossing the Kaministiquia River, at Fort William, Ontario, Canada.

The proposed terminals of the Grand Trunk Pacific at the head of Lake Superior are separated from the main line by the Kaministiquia River. At the bridge site the river has a maximum depth of about 20 ft. at low water, being at that stage 325 ft. in width. The north bank of the river, to which the draw span of the bridge swings, is steep, and rises to a height of about 35 ft. above water. The south bank is low, sloping from the river at a grade of 1 : 10.

Test borings, taken at the bridge site during the winter of 1906-07 by the railway engineers, showed a stratum of firm blue clay, about 35 ft. thick, covering a layer of water-bearing gravel varying in thickness from 3 to 7 ft. The water contained in this gravel was under sufficient pressure to maintain a steady flow through several of the abandoned

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FIG. 1.—GRAND TRUNK PACIFIC RAILROAD BRIDGE AT FORT WILLIAM, ONT. PILING AT PIVOT PIER CUT OFF AND CAPPED.

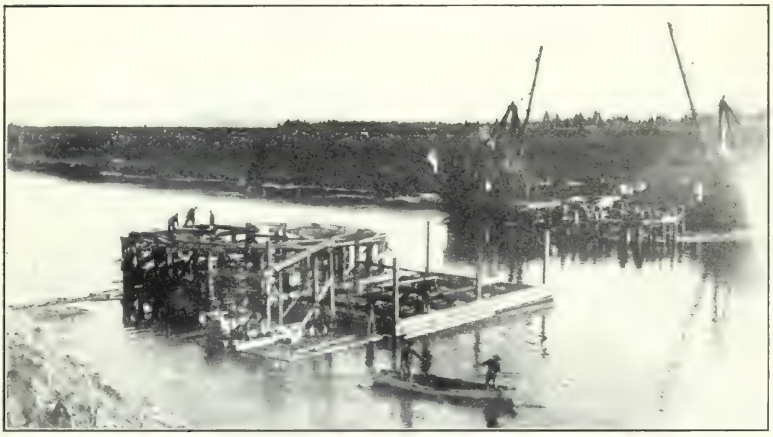

FIG. 2.—SINKING FENDER CRIB UP STREAM FROM PIVOT PIER.

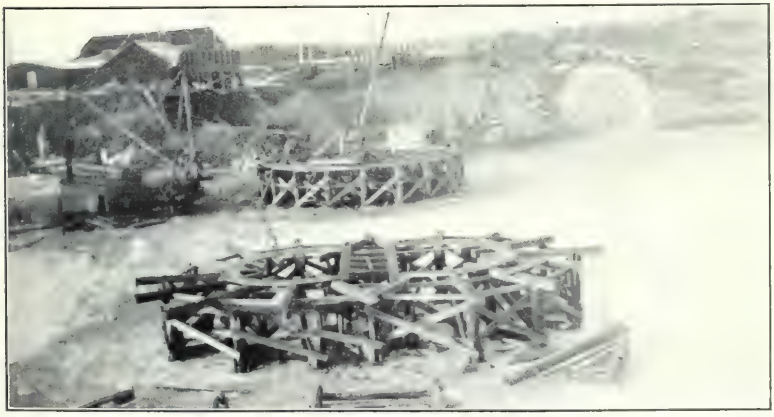

FIG. 3.—FALSEWORK OF PIERS COMPLETED.

test holes, and, when confined, it rose to an elevation 4 or 5 ft. above the river level. This gravel is underlaid with bed-rock, from 57 to 60 ft. below the water surface. The rock surface is comparatively level, dipping to the north about 2 degrees.

The plans prepared by the railway showed only the general design of the two piers, the details and construction methods being left for field decision. The pivot pier consists of a steel shell, 31 ft. in diameter, sunk to bed-rock and filled with concrete. The south pier is composed of two smaller shells, set side by side. These are 15 ft. in diameter, and 18 ft. apart from center to center.

SKETCH SHOWING FOUNDATIONS
G. T. P. BRIDGE,
FORT WILLIAM, CANADA

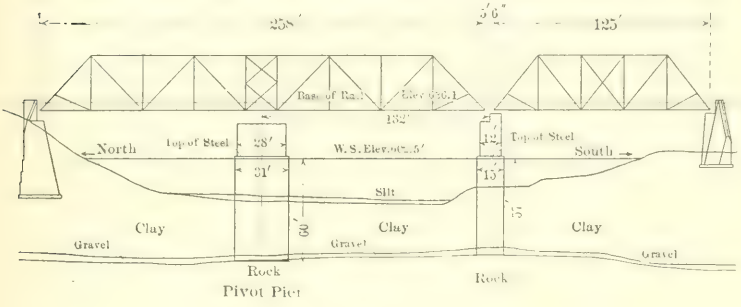

FIG. 1

The plans called for ½-in. plate, single-riveted on the horizontal seams, and double-riveted on the vertical seams, and ¾-in. rivets with round heads. The contract for the work having been let in the latter part of August, 1907, an interval of about 3 months remained before severe cold weather would set in, closing navigation and rendering all outside work more costly. Therefore operations were begun immediately.

For the falsework at the pivot pier, two rings of piling were driven about the center of the pier, the diameter of the inner ring being 37 ft., and of the outer one 54 ft. These piles were cut off and capped 8 ft. above the water. The caps were of 12 by 12-in. fir timber, radial from the center of the pier, and overhanging the perimeter of the steel shell 2 ft. There were twenty piles in each ring. These were cross-braced and tied securely, making the structure as rigid as possible, to lessen any movement that might be caused by the current, which

varied from 3 to 6 miles per hour. The piles were driven to gravel, to prevent settlement in the finished falsework. The structure was then decked and a circular track built upon it. This track was for a small derrick car, the purpose of which was to pick up the plates forming the shell, carry them to place, and hold them in position while being bolted up ready for riveting.

A slot, 2 by 8 in., was cut through the caps where their center lines intersected the perimeter of the steel shell, and a casting, through which "hanger rods" worked, was set on each cap. The hanger rods were for holding a course of plates during riveting, supporting the finished shell until the river bottom was reached, and for controlling the movements of the shell until it rested on bed-rock. The rods were 8 ft. long, of 1½-in. steel, threaded, and an eye of Swede iron was welded to the lower end of each. Hook-plates, hung in these eyes, connected the hanger rods and the plates forming the shell. There were two hanger rods at each cap, 4 in. from center to center, one outside of the shell, and one within it. The hook-plates covered four rivet holes, and were bolted to the steel plates while the derrick car held them suspended, the weight of the plates being transferred to the hanger rods by slacking the chain blocks with which the derrick car was rigged.

The plates were of ½-in. steel, 10 ft. long and 5 ft. wide; they were curved to a radius of 15½ ft., and weighed about 1 100 lb. each. There were 130 plates in the shell; 13 vertical courses of 10 plates each. The time required to swing the plates for one course from the storage yard to the derrick car, and to bolt them up ready for riveting, averaged about 2½ hours, or 15 min. for each plate. Five courses were riveted up when the shell touched bottom. The weight of the shell caused it to penetrate the bottom to an average depth of 1 ft. A 2-in. water-jet was then rigged from a single-cylinder, 6-in. steam force pump, and by this means the material at the cutting edge was disturbed sufficiently to permit a further settlement amounting to about 1 ft.

Each outside hanger rod was then carried back to the pile carrying the cap from which the rod was suspended, and bolted to the pile, thus tying the inner end of the cap in place. Jacks were placed under each cap, resting on the edge of the shell, and the combined action of the water-jet and twenty jacks forced the shell about 5 ft. farther into the bottom, the total penetration being at this time 7 ft.

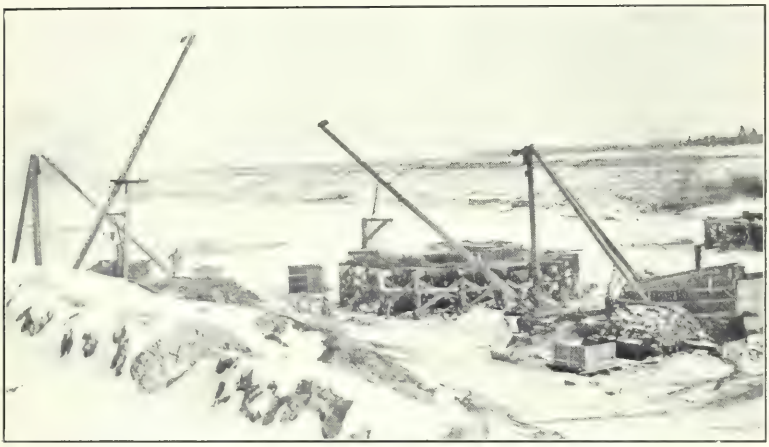

FIG. 1.—PIVOT PIER AND DERRICK.

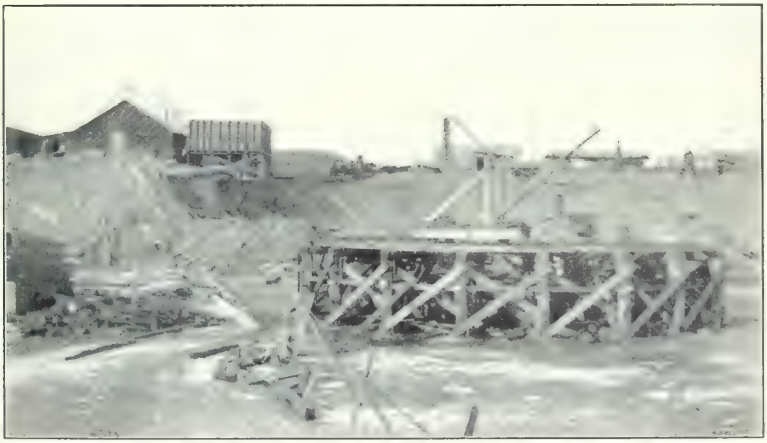

FIG. 2.—FALSEWORK AND DERRICK CAR, PIVOT PIER.

The shell was then pumped out. As the water was removed, a course of bracing was placed at each horizontal seam of the shell. This bracing was of 12 by 12-in. hemlock, four timbers being set in the form of a square. Two squares were placed together, one being turned 45° about the center of the pier, thus forming a star-shaped set of bracing, giving eight points of support to the sides of the shell, and leaving an octagonal central opening about 18 ft. across.

Five days after the shell was cleared of water, the material under the cutting edge gave way, and the shell filled, the blow being confined to about 4 ft. of the perimeter. This hole was carefully filled, and the shell pumped out again, only to blow out, about 5 hours later, at a different point.

At this time a flow of water within the shell was noticed, coming apparently from one of the holes left by the test borings, and being approximately equivalent to the flow from a 4-in. pipe. It was evident that there would be great difficulty in keeping the shell cleared of water, and an attempt was made to sink it still farther into the river bed. It was loaded with 175 tons of steel rails, the jacks were placed in position as before, and it was forced downward another 5 ft., the total penetration then being 12 ft. Another attempt to clear the shell of water, resulting in failure, indicated that the flow from the gravel had destroyed the integrity of the material within the shell. This method was abandoned and the work proceeded as follows:

The overhanging caps were cut off, and two more courses of steel were riveted up. Timbers were placed across the top of the shell, and a tight floor, about 1 ft. from water level, was hung from these timbers, within the shell. A form was built, following the octagonal outline of the inner opening left by the timber bracing, and concrete was deposited between this form and the steel shell. Meanwhile, a derrick was erected on a fender crib which had been sunk immediately up stream from the pivot pier, and excavation was commenced within the shell, using an orange-peel bucket. The settlement of the shell was constant from this point, and, as it sank, the ring of concrete was carried upward, thus forming a steel-protected pipe or drum, having an outside diameter of 31 ft., the inner opening being octagonal.

When the shell finally reached rock, the surface of the rock was cleaned, and a layer of concrete about 7 ft. in thickness was deposited under water, which sealed the flow from the water-bearing gravel and

allowed the removal of the bracing. When the timber was removed, the interior opening was filled with concrete, forms were built at the water level, and the pier was carried up to its final elevation.

The concrete used was a 1:3:5 mixture, the gravel and sand being brought in by train from a point thirty miles distant on the line of the Grand Trunk Pacific. This material was clean, and in its natural state closely approximated the required proportions. Tests were made from time to time to determine its condition, and enough sand was added to make it comply with the specifications. Screening was considered unnecessary. To prevent the concrete from freezing, the sand and gravel were heated. Three heaters, made from sections of 36-in. cast-iron pipe, were placed near the stock piles. Wood for heating purposes was plentiful, and the cost of heating all material averaged about $1.00 per cubic yard of concrete. The materials were conveyed to the mixer by cars, and the concrete was swung to the derrick on the fender crib by using a second derrick located on the river bank. After the ice formed, the derricks exchanged boxes on the ice, and the mixer, running constantly, kept one batch of concrete moving at all times. The batches averaged 22 cu. ft., the average output being 11 cu. yd. per hour.

In the concrete, pieces of stone, containing from 2 to 15 cu. ft., were placed by the derrick on the fender crib. These stones were heated by immersing them in a tank of hot water for a few minutes, the volume being indicated by a displacement scale marked off inside the tank. In this way a check on the total volume of material was obtained, the combined volumes of stone and concrete equaling the quantity indicated from measurements made inside the shell. The total quantity of stone and concrete in this pier is 2 200 cu. yd.

Conditions at the smaller pier on the south side of the river were somewhat different, the water being only 17 ft. deep, and bed-rock 57 ft. from the water surface.

This pier consists of two concrete-filled steel shells, 15 ft. in diameter, set side by side at right angles to the center line of the bridge, with a space of 3 ft. between them. Falsework, similar to that at the pivot pier, was erected. To handle the steel plates, a derrick was placed on the center line of the bridge, resting on three small cribs sunk for the purpose.

The shells were lined with concrete as they sank, and, owing to the

FIG. 1. INTERIOR OF SHELL OF PIVOT PIER, SHOWING BRACING.

FIG. 2.—COMPLETED PIERS, LOOKING UP STREAM FROM THE SOUTH BANK.

fact that they could be pumped out, about two-thirds of the total excavation was completed in the dry, the derrick disposing of the material as it was excavated. The shells were in no way connected, but were carried down with as nearly the same amount of progress as possible. The general progress was in jumps of from 2 to 5 ft. The steel was carried high enough at all times to prevent any chance of the shell disappearing beneath the surface. The progress could be closely controlled by the amount of excavation. The shells were brought to place finally with a difference in elevation of less than ½ in. Along the center line of the bridge, they were about 4 in. out of position. This was corrected when the forms were built at the water line, where a slight offset had been provided for such a contingency.

There are 1 055 cu. yd. of concrete in this pier, the same mixture, 1:3:5, being used as in the pivot pier. The two shells were joined at the top by a reinforced concrete beam. The sand and gravel for this pier had been placed on the south bank of the river before the close of navigation. This material was dredged from the bed of Lake Superior during the summer, and was used without screening. The mixing plant for this pier was located on the south bank, and the concrete was conveyed to the derrick by cars running on a track laid on the ice.

It was thought that difficulty might arise from the shrinkage of the concrete inside the shell, with a consequent destruction of the bond between it and the steel, and, to prevent this, the first course of concrete was allowed to set under water, where some expansion would occur. The concrete in the piers was kept warm, where exposed to the air, by covering the piers and turning live steam into the enclosed space. The concrete below the ice line, and under water, was, of course, in no danger from freezing.

The steel shells were rendered water-tight by caulking them at the laps, and by driving a small triangular wedge at the scarfs. The resistance of the material through which the shells were sunk was found to vary from 275 to 320 lb. per sq. ft. This is the net skin friction, after deducting the resistance caused by the cutting edge, the laps in the plates, and the rivet heads.

The cutting edges were reinforced by riveting, to the lower course of steel, an additional ½-in. plate, 3 ft. wide. This was considered sufficient, as the test borings had shown the material to be clear, and free from sunken trees or boulders.

The riveting and caulking were done with air, the compressor being of the locomotive type. Owing to the atmospheric moisture, the air line and hammers at times gave some trouble, due to the intense cold.

The plates for the shells were punched and bent at Chicago and shipped ready for erection. They were fitted up easily and rapidly, as the work of bending and punching had been done well and carefully.

Work was carried on as rapidly as possible, three shifts being employed on the concrete at times. The total time consumed in this work, disregarding lapses and minor delays, was about 5 months. The cost of the two piers was about $60 000.

The work came under the supervision of Mr. George A. Knowlton, Division Engineer in charge of the Lake Superior Branch of the Grand Trunk Pacific Railway. The writer designed the lay-out of the plant and the method of construction, and was in charge of the work.

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THE HYDRO-ELECTRIC DEVELOPMENT AND TRANSMISSION LINES OF THE CANADIAN NIAGARA POWER COMPANY.

By A. H. Van Cleve, M. Am. Soc. C. E.
To be Presented October 21st, 1908.

History and General Scope of the Development.

As soon as the hydro-electric development of The Niagara Falls Power Company, at Niagara Falls, N. Y., was fairly under construction, the attention of capitalists was attracted to the water-power possibilities of the Canadian side of the Niagara River. The conditions for development there differed materially from those on the American side, owing to the fact that all the land adjoining the Horseshoe, or Canadian, Falls was and is owned by the Province of Ontario, being reserved as a public park and administered by a Park Commission. As is evident by an inspection of Fig. 1, the Queen Victoria Niagara Falls Park embraced all the most desirable sites for power-plants, and, as the entire lower bank of the gorge from the Falls to a point near Lake Ontario was included in the Park Reserve, it was evident that a hydro-electric development could take place only by arrangement with the Park Commissioners.

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Accordingly, on April 7th, 1892, an agreement was entered into between Messrs. Albert D. Shaw, Francis Lynde Stetson, and William B. Rankine (representing a company thereafter to be incorporated), and the Commissioners for the Queen Victoria Niagara Falls Park, granting exclusive rights for taking and using the waters of the Niagara River within the limits of the Park, excepting as certain minor grants had previously been made. The agreement provided for the payment of certain rentals, which increased from year to year and were independent of the amount of power developed.

In view of the present state of the art, it is interesting to note that the company was granted the right to generate and transmit pneumatic power as well as electricity, thus showing that, in the minds of the incorporators, at least, the success of the generation and transmission of electric power was not so assured as to dispense with a provision for other forms of power.

The agreement further provided that, on or before November 1st, 1898, the company should complete water connections for the development of 25 000 h. p., and should at that date have ready for use and transmission 10 000 h. p. The agreement with the Park Commissioners was ratified by the Legislative Assembly of the Province of Ontario on April 14th, 1892. The company formed to carry on the power development was known as the Canadian Niagara Power Company.

By reference to Fig. 1 it will be seen that there was a body of water separated from the rapids above the Falls by Cedar Island. In the original plans it was proposed to deepen this natural channel and to connect it with the river by a wide mouth at the upper end of Cedar Island. Such canal was to supply water to three successive developments of 50 000 h. p. each, two tunnels being provided for discharging the water north of the Horseshoe Falls. There were numerous engineering objections to the plan proposed, and it is, indeed, doubtful whether a plant thus located could have been operated successfully during adverse ice conditions. In locating and designing a power-plant in a beautiful park, æsthetic, as well as engineering, conditions must be conformed to, and such conditions affected not only the site originally selected but also that finally adopted, the question of harmony with the surroundings being ever present in the minds of the designers.

No construction took place at the point described. Many of the

FIG. 1.—PUTTING PUDDLE IN COFFER-DAM.

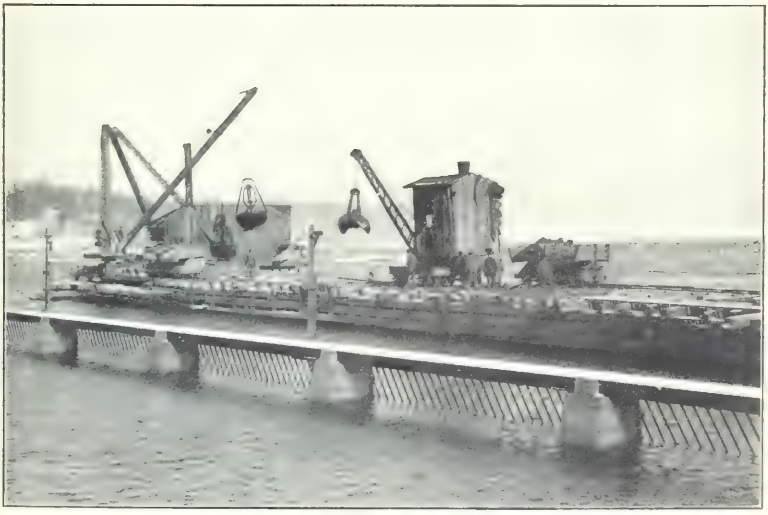


FIG. 2.—PROGRESS OF REMOVAL OF COFFER-DAM.

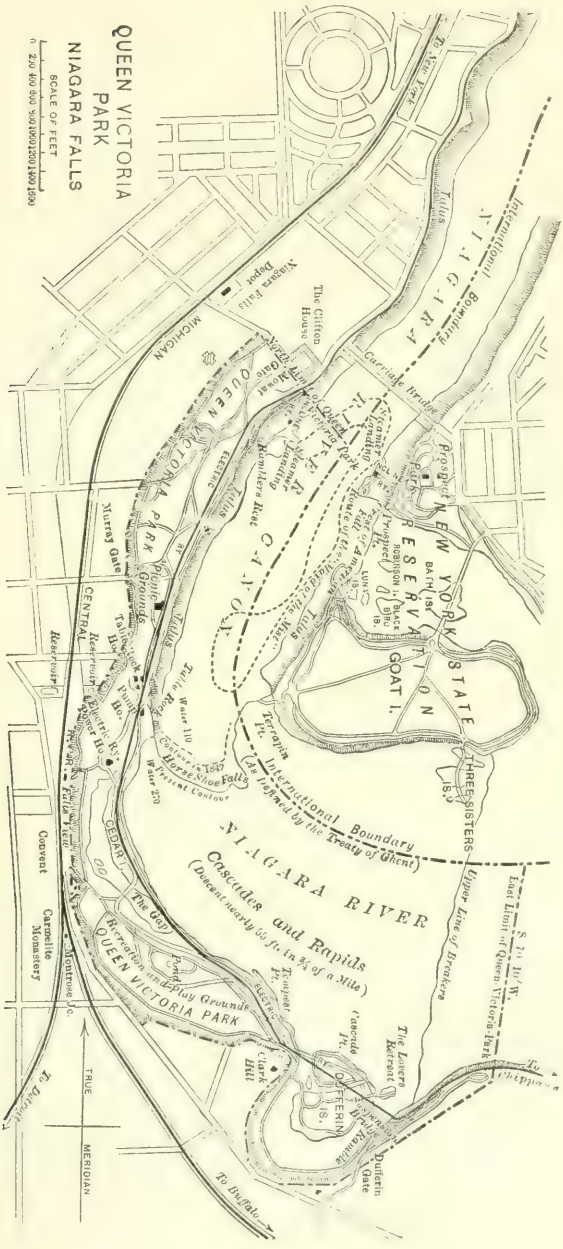

QUEEN VICTORIA PARK
NIAGARA FALLS
SCALE OF FEET
Niagara Falls Depot
The Clifton House
MICHIGAN
QUEEN VICTORIA
Victoria Gate
Electric Railway
Table Rock
Murray Gate
Refreshment Rooms
Terrapin Pt.
Horse Shoe Fall
Water 110
Water 270
Present Contour
CANADA
Terrapin Pt.
International Boundary
(As Defined by the Treaty of Ghent)
NIAGARA RIVER
Cascades and Rapids
(Descent nearly 55 ft. in ¾ of a Mile)
Pt. Turpin
Cascade Pt.
Convent
Carmelite Monastery
Montrose Jc.
Recreation and Play Grounds
QUEEN VICTORIA PARK
Clark Hill
The Lovers Retreat
Queen's Pt.
Victoria Gate
Bulletin Gate
To Buffalo
TRUE MERIDIAN
Suspension Bridge
Prospect Park
NEW YORK STATE RESERVATION
GOAT I.
THREE SISTERS
Upper Line of Breakers
East Limit of Queen Victoria Park

FIG. 1.

capitalists were interested both in the Canadian Niagara Power Company and in The Niagara Falls Power Company, and, during the years from 1890 to 1900, the latter company was working out many novel problems in hydro-electric development, and it was desired that the Canadian company should take advantage of all the experience to be gained in the design and operation of the American plant.

The period referred to in the original grant thus elapsed with only a nominal amount of work being done, and, on July 15th, 1899, a supplemental agreement was entered into between the Canadian Niagara Power Company and the Park Commissioners which rescinded that clause of the former agreement which made the company's rights for the use of water a practically exclusive one, made the rental partially dependent upon the amount of power generated, and extended the time for the completion of the first development to July 1st, 1903. The time at which the company was to have ready for use, supply, and transmission 20 000 h. p. was afterward extended by agreements and legislative enactments to January 1st, 1905, and the requirements were complied with at that date.

The agreement of July 15th, 1899, provided in detail that not only the designs of the works of the company but also the manner of their construction should be subject to the approval of the Park Commissioners, and the document was accompanied by the map shown on Fig. 2, indicating the point at which the first development was to take place, as well as the general character of its design. The dotted lines on Fig. 2 indicate the outlines of the slough back of Cedar Island. It will be noted that the works have been moved up the river from the site originally selected. The length of the discharge tunnel was thus increased, but the form and location of the inlet canal were better suited to meet ice conditions. But small saving could have been effected by utilizing the natural watercourse, as it was shallow. It is evident that the ideal location for the power-house was on the shore of the river opposite its present position, but objections were made to such a site on the ground that a building at that point would interfere with the view of the rapids.

The plan shown on Fig. 2 contemplates an ultimate development of 100 000 e. h. p. by twenty units of 5 000 e. h. p. each, with two spare machines. This plan was one of the general drawings accompanying the agreement of July 15th, 1899, previously referred to. Thus far

FIG. 1.—PROGRESS ON OUTER ICE RACK

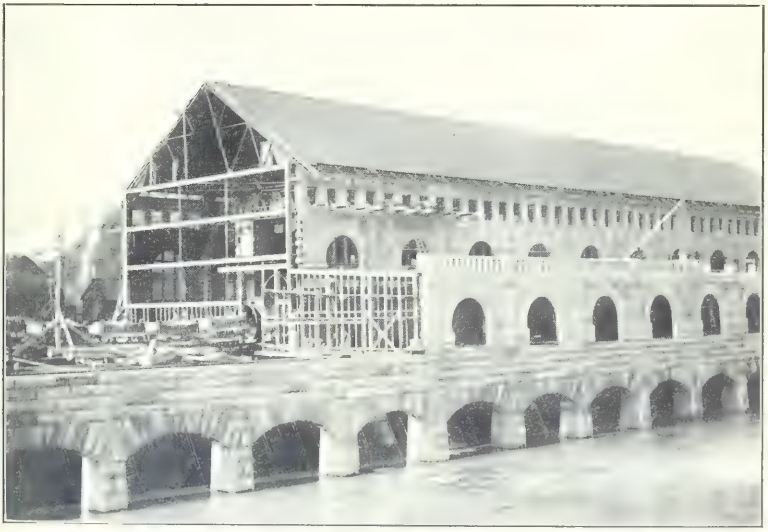


FIG. 2.—TEMPORARY SOUTH END OF POWER-HOUSE.

in the course of the design, units of a capacity greater than 5 000 e. h. p. had not been contemplated, but the state of the art had advanced sufficiently so that in 1901 inquiries submitted to manufacturers of both hydraulic and electrical machinery elicited favorable replies as to their ability to design and furnish machinery for units of 10 000

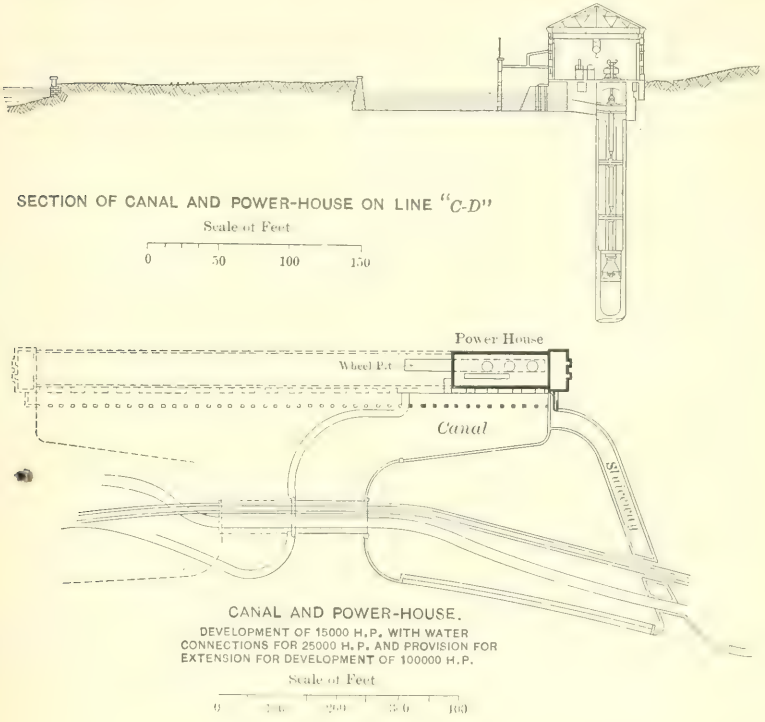


FIG. 2.

e. h. p. The larger unit was adopted for the final design, the following being among its advantages:

(*a*)—Somewhat lower cost of hydraulic machinery per horse-power developed;

(*b*)—Materially lower cost of electrical machinery per electric horse-power developed;

(*c*)—Decreased length of canal, wheel-pit, and power-house, thus effecting a large economy in the cost of excavation and masonry;

(*d*)—Opportunity for increased development within the area in the Park assigned to the Canadian Niagara Power Company;

(*e*)—Convenience and economy of operation due to the reduction in number of units, it being remembered that the bus-bars of the plant under consideration were to be connected with those in the power-houses of The Niagara Falls Power Company, where 5 000-e. h. p. units were available to subdivide the load.

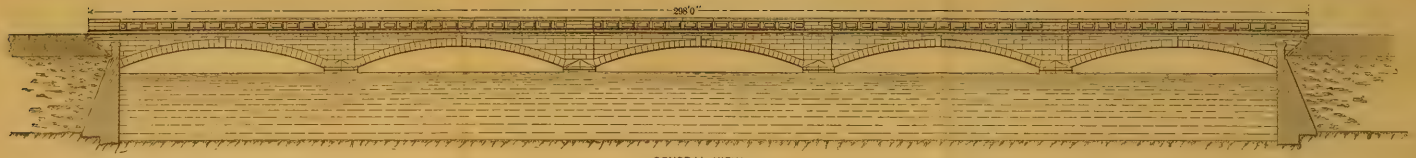
The increase in the size of the units, of course, resulted in changes in the entire design of the plant, and on Plate LXXVI is shown the general arrangement finally adopted.

At the time of the agreement of July 15th, 1899, it was the intention to provide water connections for 25 000 h. p. only, and to excavate a wheel-pit for only five units of 5 000 e. h. p. Before any contracts were let, it was decided to make water connections for 50 000 h. p. and to excavate a wheel-pit 250 ft. long, two units of 10 000 e. h. p. to be installed. Finally, on February 2d, 1903, a contract was let for the completion of the canal and wheel-pit to their full capacity. The plant, as now constructed, consists of a canal, wheel-pit, and tunnel having a designed capacity of 100 000 e. h. p., an installation of five units, each having a nominal capacity of 10 000 e. h. p., and a power-house sufficiently large to enclose them.

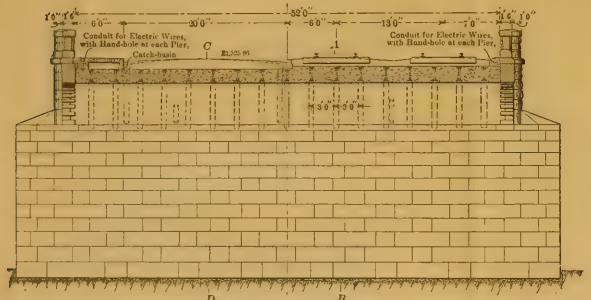
Having given the foregoing introductory remarks in regard to the history and general scope of the development, the writer will now describe the more important features of the several parts of the plant.

Coffer-Dam and Canal.

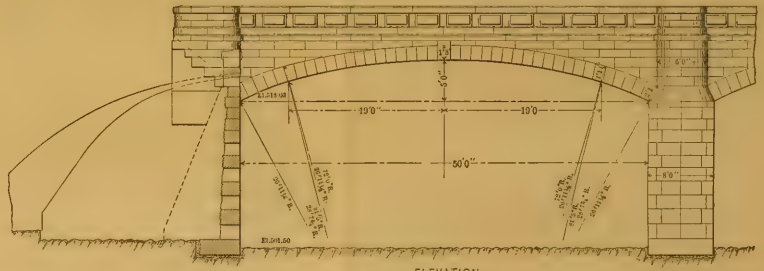
In order to obtain a sufficient depth of water at the entrance to the canal, it was evident that a portion of the river bottom would have to be unwatered, that the necessary excavation might proceed. It was a perilous matter to obtain in the rapids the soundings which were requisite for a determination of the limits within which such excavation was necessary, but, after several narrow escapes in the turbulent current leading to the Horseshoe Falls, the necessary soundings were obtained and a coffer-dam constructed to enclose the desired area, a depth of 15 ft. of water being secured where possible.



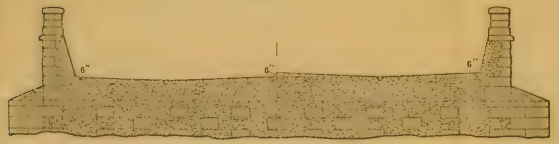
GENERAL VIEW



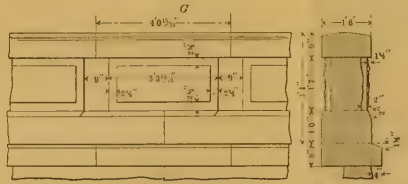
SECTION AT CROWN OF ARCH



ELEVATION



SECTION AT CENTER OF PIER



DETAIL OF BALUSTRADE

SECTION *G-H*

STONE BRIDGE OVER CANAL INLET

CANADIAN NIAGARA POWER COMPANY

The coffer-dam consisted of two rows of rock-filled cribs with a puddle space between, additional tightness being secured by double sheeting and a concrete toe on the inside. Fig. 1, Plate LXXIV, shows the coffer-dam at the stage when the puddle was being placed, and, as shown in this photograph, the turbulence of the water gives an idea of the difficulties encountered in its construction.

The removal of the coffer-dam proved almost as difficult a problem as its construction. The progress made by the contractor was so slow that, in view of the necessity of operating the plant, the engineers had to assume control of this work. The method finally adopted is illustrated in Fig. 2, Plate LXXIV. The rock was removed to the water line by hand, the puddle was dredged by an orange-peel bucket attached to a Brown hoist, and the turnbuckles uniting the cribs were blown apart by dynamite. The upper timbers were removed, the cribs were racked by light charges, pulled apart by the heavy traveling derrick shown at the left, and the remaining material was dredged with the extra heavy orange-peel bucket. Soundings determined when the original surface was uncovered. Unusually severe winter weather added materially to the difficulty of the work.

Plate LXXVI shows the form and dimensions of the finished canal and its relation to other parts of the plant. The water areas are selected so that, when 100 000 h. p. is being generated, the average velocity, with the water at low stage, will be 2.7 ft. per sec. at the entrance, 3 ft. per sec. under the bridge, and 2.35 ft. per sec. through the submerged arches separating the forebay from the main canal.

Bridges were necessary to carry a driveway and a double-track electric line across the canal and ice sluiceway. Occupying a prominent position in the Park, it was desirable that the bridges should present a pleasing appearance, and reinforced concrete arches faced with limestone were adopted, there being five spans of 50 ft. each at the canal crossing. No special difficulties were encountered in the construction of this bridge, and, notwithstanding the small ratio of rise to span (1:10), the settlement was negligible.

The position of the plant below the principal cascades of the river makes the ice conditions difficult, as the Niagara River carries large quantities of ice during the greater part of the winter. Normally, the most of it passes close to the northerly side of the river, but when the wind is easterly and northerly large quantities are carried to the

Canadian side, and, in its progress down the rapids, is finely broken and, by the turbulence of the water, is carried to a great depth below the surface. Under certain conditions true frazil is abundant, but is far more prevalent on the American than on the Canadian side. The plant is protected against ice in the following manner:

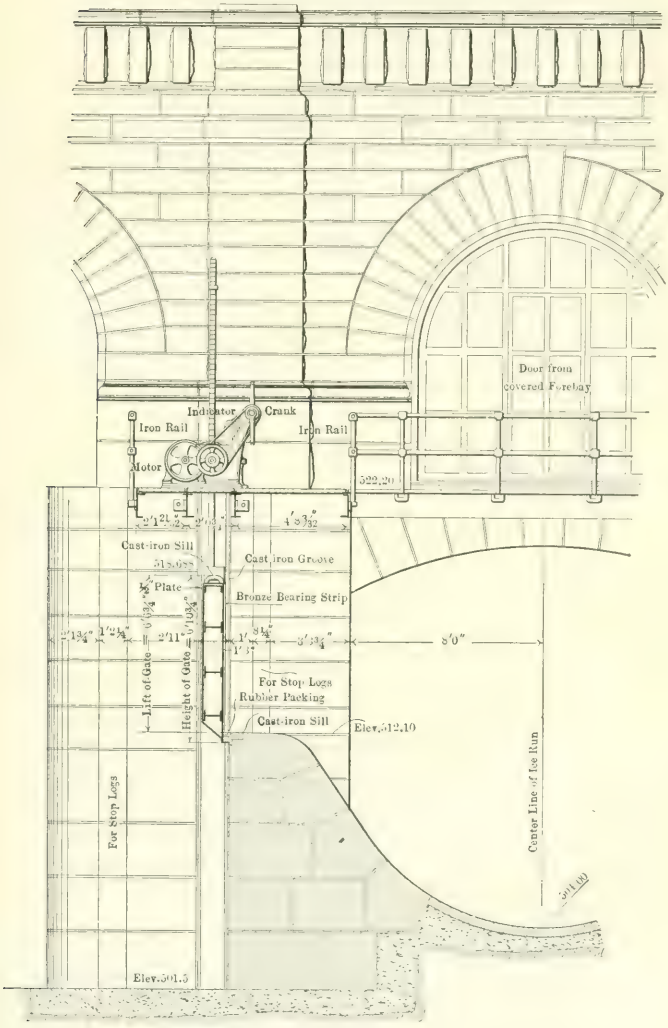
(*a*)—By a rack of 2-in. inclined bars, spaced 12 in. apart, connected with a steel framework carried by the masonry piers at the entrance to the canal, such piers being shown at the bottom of Fig. 3. This rack extends 4½ ft. below the normal water surface, and was originally intended to ward off only large cakes of ice. For the purpose of keeping out fine ice, steel plates were afterward attached to the rack, and, although only partly successful in accomplishing their purpose, produced an unlooked-for result. The water outside of the rack is in a state of violent disturbance, the waves being about 5 ft. in height. Before the attachment of the plates, the rise and fall of the water surface inside the rack was the same as that outside, but the plates naturally produced a stilling effect and the river surface was alternately higher and lower than that in the canal. This alternation of relative levels had the effect of a constantly recurring series of blows upon the piers to which the rack is attached. Although the several courses of masonry were connected with two 1½-in. rods, these were in some cases broken, and the upper portion of the piers tilted inward. It has become necessary to strengthen the piers by the addition of two 2-in. rods carried from the top into the ledge rock 2 ft. below the canal bottom. Fig. 1, Plate LXXV, clearly shows the rack as originally constructed and the bridge previously referred to. Afterward, a railing and electric light poles were added to the footwalk.

(*b*)—The outer wall of the forebay superstructure rests on a wall of first-class masonry containing a series of arched openings 19 ft. 9 in. wide, the crown of each arch being 2.1 ft. below low water or 4.8 ft. below mean water. Two openings are provided for each unit. The edges of the openings are rounded, and their sides are smooth-pointed. Fig. 2, Plate LXXV, a photograph taken near the close of the construction period, shows the character of the masonry arches and their relation to the power-house and forebay. This arrangement of submerged openings effectually prevents the entrance into the forebay of floating ice and such other floating material as may find its way into the canal.

FIG. 1.—SOUTH END OF WHEEL-PIT UNDER CONSTRUCTION.

FIG. 2.—TUNNEL PORTAL UNDER CONSTRUCTION.

SLUICE GATES FOR ICE-RUN
CANADIAN NIAGARA POWER COMPANY
PLAN SHOWING GENERAL ARRANGEMENT



SECTION LOOKING WEST

FIG. 3.

(*c*)—It was evident that some means must be provided for removing from the canal such finely-divided ice as might pass the outer rack, as otherwise it would accumulate in the canal and, by adhesion to the underside of the surface ice, reduce the water area materially. If allowed to accumulate to a depth below the tops of the arches, it would be carried into the forebay. The construction of a shaft connecting with the outlet tunnel would have been expensive, and to have disposed of the ice in this way would have reduced the tunnel area available for discharge water from the turbines and would have introduced serious cross-currents in the tunnel. Fortunately, the rapid descent of the river below the mouth of the canal enabled a discharge channel to be built, as shown on Plate LXXVI. A difference of level of about 2 ft. is thus obtained between the surface of the water in the sluiceway and that in the canal. The sluiceway has a width of 16 ft. and a depth of water of about 10 ft. It is separated from the canal by masonry weirs, the tops of which are 2.2 ft. below low water. A further separation is effected by steel gates sliding in cast-iron grooves, as shown on Fig. 3. Two openings, each 16 ft. in width, provide for the flow of ice from the canal, while a third gate of similar construction allows the escape of any material which may enter the forebay. The gates are operated by hand-power, although provision is made for the attachment of motors.

(*d*)—A steel rack, composed of 3 by $\frac{3}{8}$-in. bars, $1\frac{13}{16}$ in. apart, making an angle of 60° with the horizontal, extends the entire length of the forebay, thus protecting the entrances to the penstocks. Its construction calls for no remark except the statement that it is in three sections vertically, the middle sections, 3 ft. 4 in. wide, sliding in grooves and adjustable in any desired position. The supporting framework was designed to resist a water pressure of 2 ft., it being considered that a sudden flow of frazil might clog the racks before they could be cleared.

The excavation of the canal involved the removal of about 14 ft. of sand, clay, gravel, and boulders, and from 1 to 3 ft. of ledge rock. The total quantity of earth excavated from the canal and wheel-pit was 85 000 cu. yd. Most of this earth excavation was well adapted for steam-shovel work, but that method was used by the contractor for only a small part of the work, most of the material being removed by pick and shovel and hauled to the dump in 1-yd. wagons.

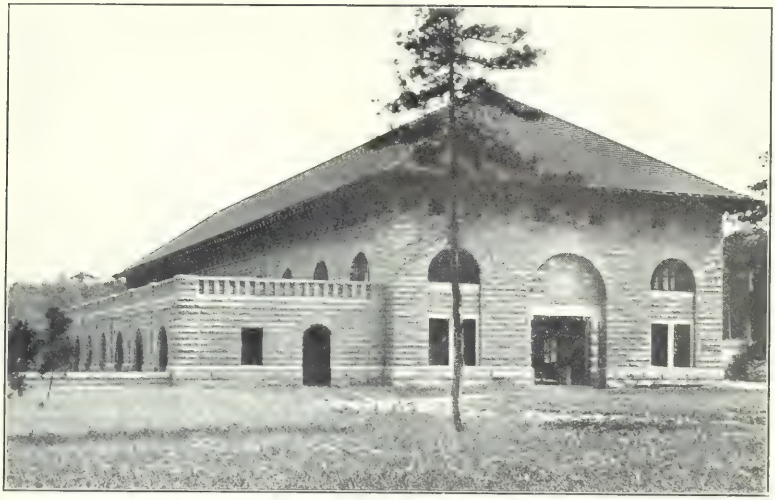


FIG. 1.—NORTH END OF POWER-HOUSE.

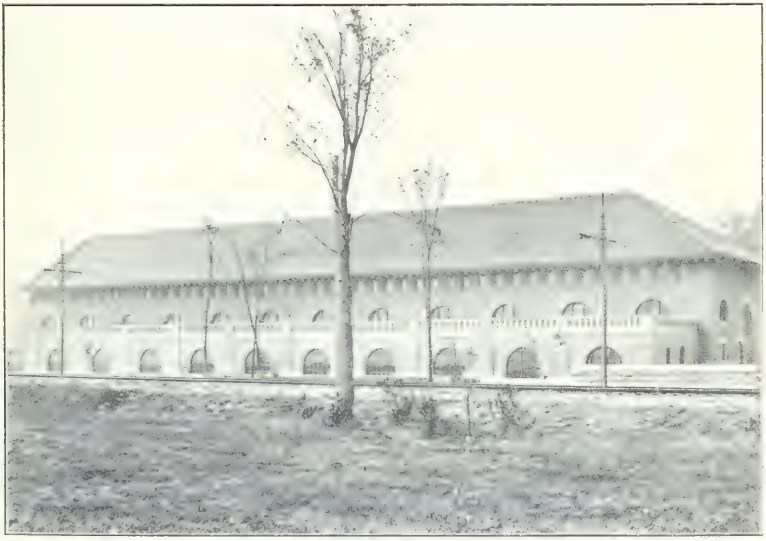


FIG. 2. VIEW OF POWER-HOUSE FROM THE EAST.

Wheel-Pit.

Although the general location of the plant predetermined the generation of the power by wheels placed in an excavation and discharging through a tunnel having an outlet below the Falls, yet the form of such an excavation was dependent on the question of vertical or horizontal shafts for the machinery. This problem was decided in favor of vertical shafts, not only because of the greater convenience in operation afforded by having the generators on the power-house floor, but because of the practical difficulties which would be encountered in arranging the machinery for easy dismantling if horizontal shafts were used. The fact that no excavations of similar depth had been made on the Canadian side of the river introduced an uncertainty as to whether a wheel-pit could be made sufficiently dry to maintain electrical machinery successfully therein. On account of these and other considerations, it was determined to use units with vertical shafts, the spacing of the units to be 48 ft. The length of the wheel-pit was fixed at 566.84 ft. and the depth, to meet the tunnel grade, 171.1 ft. below the power-house floor, the width of rock excavation being 20.5 ft. The maximum depth of rock excavation was about 153 ft., the total quantity of rock excavation in the wheel-pit proper being about 226 000 cu. yd. To insure close adherence to the limits of excavation, and to prevent the shattering of the rock, and consequently the increased flow of water, the sides and ends of the pit were channeled. On account of the clearance requirements of the channelers, the cuts were made on an angle, a 6-in. offset being provided at the bottom of each cut of 6 ft. The rock was a limestone of varying character for two-thirds of the depth, and the lower third was shale.

Plate LXXX shows the completed wheel-pit, with the machinery installed, and also the lines of excavation and the brick lining walls. Above the turbine deck the average thickness of the lining walls is 15 in., and below that level it is 24 in., vitrified brick being used in the invert and a portion of the side walls. Fig. 1, Plate LXXVII, shows the rock walls, and the channelers at work, the pit having been excavated for about one-third of its depth when the photograph was taken. The brickwork seen in the upper left-hand part of the view is that built along the outside of a longitudinal recess cut into the rock at a water-bearing seam in order to collect the water. All the floor beams and girders shown on Plate LXXX rest on castings built into

the side walls; the openings for these castings were gadded and broached. Excavations for the draft-tubes were treated in a similar manner. As the latter recesses were 51 ft. by 10 ft. 6 in. by 7 ft. 10 in., and were in close proximity to other checks, their excavation was a matter of no little difficulty. Four chambers, 21 ft. wide, 50 ft. long, and of varying depths, were cut into the east side of the wheel-pit to provide for the installation of auxiliary machinery, such chambers being lined with brickwork 2 ft. thick.

The excavation of the wheel-pit was let in two contracts, the first covering a length of only 270 ft. The final blasting in the second section of the pit was not completed until July, 1905, or 6 months after the plant was put in operation. It was necessary, therefore, to take great care, in blasting in the extension, to avoid disturbance of the alignment of the machinery. The movement of the wheel-pit walls was also increased by the excavation in the extension, and, although no serious results followed, the conditions, for the time being, were very undesirable. As the tunnel work was in advance of the wheel-pit excavation, an effort was made to save time by carrying it under a part of the wheel-pit. This excavation shattered the side walls, and it is questionable whether anything was gained by this method.

The bottom of the wheel-pit was given a grade of 3% in order to equalize as fully as possible the velocities in the discharge water in the wheel-pit. To preserve an absolutely uniform velocity, with all the wheels in operation, would require that the width of the invert should decrease toward the south end, but such a design would have involved many complications not justified by the advantage to be gained.

The upper part of the wheel-pit, to a depth of from 25 to 30 ft. below the power-house floor, is lined with a retaining wall, the upper portion of which is faced with first-class, close-pointed masonry, while the lower part is of concrete. In connection with this upper lining wall, were built the arches, 24 ft. in length, supporting the stationary part of the generator. These may be seen on Plate LXXX, while Fig. 1, Plate LXXVII, shows the character of the construction prior to pointing.

The six floors in the wheel-pit require little comment; there are also platforms around the shaft at four other levels. Access is thus provided for the inspection and repair of all bearings and shaft

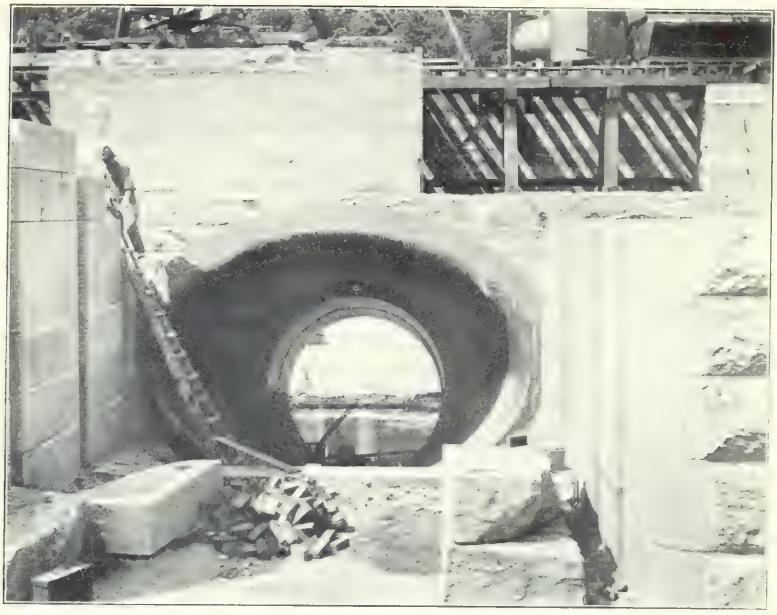


FIG. 1.—BRICK RING AROUND NO. 1 PENSTOCK MOUTHPIECE.

FIG. 2.—INTERIOR OF POWER-HOUSE, SHOWING ENCLOSURE FOR OIL SWITCHES.

couplings. The principal decks are covered with "Never-Slip" plate; the platforms and lower deck are composed of rack-bars. All main cross-beams are carried on castings, to which they are attached in such a way as to permit the freedom of motion required by the movement of the wheel-pit walls.

Tunnel.

The general alignment of the tunnel is shown on Plate LXXXI, which also indicates the completed plant and its relation to surrounding objects. The southerly tangent was of necessity in the prolongation of the wheel-pit, and it was also desirable that the alignment at the portal should be approximately at right angles to the face of the cliff. The wheel-pit of the Niagara Falls Park and River Railway was to be avoided, and the portal could not be placed beneath its discharge tunnel. The rock formation at the point selected seemed most satisfactory. These conditions resulted in a tunnel 2 164 ft. long, the alignment consisting of 48% tangent and 52% curvature with a radius of 650 ft. The location of the portal and its relation to the Falls is shown by Fig. 2, Plate LXXVII, and Fig. 1, Plate XCI, which serve to give a very clear impression of the difficulty of carrying on work at this point. Owing to the prevailing direction of the wind, the heavy spray from the Falls fell on the work almost constantly; ice commenced to form in December, and lasted until May, and high water covered the work almost completely at times. The tunnel cross-section finally adopted was of horse-shoe form, 25 ft. high and 18.84 ft. wide at its widest point.

The original intention had been to use four rings of brickwork throughout, but, as built, they were used for the arch only, the remainder of the lining being of concrete with only a facing of hard brick. Doubtless some advocates of concrete would have been willing to dispense with the brick entirely, even with tunnel velocities of 30 ft. per sec., but the perfect condition of the brick lining in the tunnel of The Niagara Falls Power Company after seven years of service was a strong argument in favor of the use of brick.

The gradient of the tunnel is 7 ft. per 1 000, except near the portal where it falls 11.2 ft. in 103.08 ft., a reverse vertical curve, with a radius of 248 ft. at the springing line, being used. The greater part of this portion is lined with granite blocks 2 ft. thick to withstand the increased velocity of the water.

After the removal of the coffer-dam between the portal and the river, it was found necessary to deepen the water channel. This was done in the summer of 1905 by an orange-peel bucket attached to a derrick, liberal quantities of powder being used.

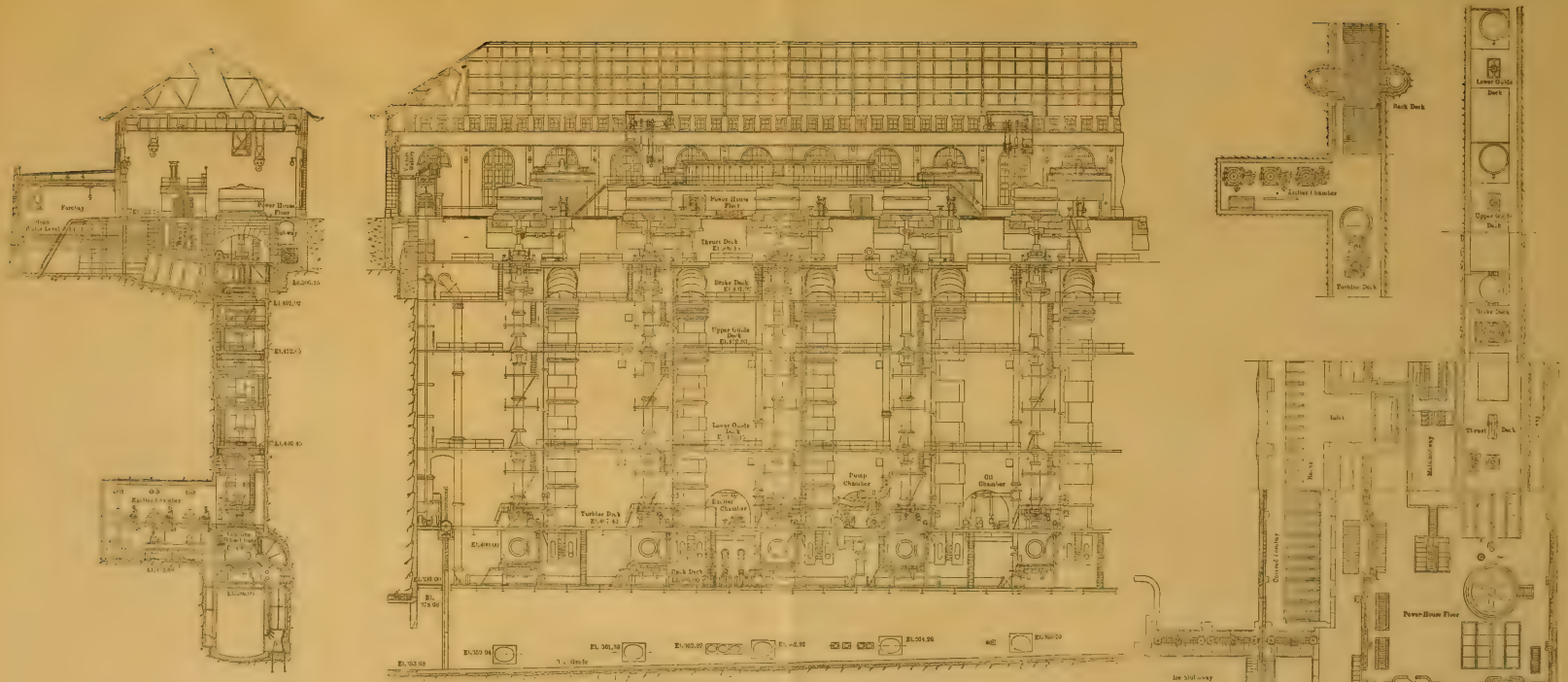
The excavation of the tunnel was carried in both directions from a shaft 1 470 ft. from the north end of the wheel-pit, and was also carried in for some distance from the portal. The section was removed as a heading and two benches, the timbering following the excavation of the heading as closely as possible. The excavated material was principally shale, which disintegrated rapidly upon exposure to air and moisture. Most of it was used in filling the lagoon adjoining Cedar Island.

The availability of the remainder of the material excavated from the tunnel, as well as the larger quantities removed from the canal and wheel-pit, enabled the Park Commissioners to consummate a plan which they had formed many years before and had partly carried out in 1895, namely, the reclamation of a large area of unsightly rock surface at the west end of the Horseshoe Falls left practically bare by the rapid recession of the apex of the falls and the consequent deepening of the central portion of the channel.* The filling of this area made possible the construction of walks and driveways from which visitors may obtain a close view of the most impressive portion of the cataract, and has greatly improved the scenic conditions at this point.

Power-House.

The power-house superstructure is a fire-proof building, 110 ft. 10 in. wide, now completed for a length of 296 ft., to cover five units, its ultimate length being 588 ft. 8 in. The main power-room, covering the generators and switch-board, has a clear width of 68 ft. 2 in. and a height of 64 ft. 1 in. to the ridge. Along the east side of the building is the enclosed forebay, having a clear width of 36 ft. 6 in., the clearance line of the roof girders being only 14 ft. above the floor. This portion has a roof which is nearly flat, and is surrounded by an ornamental parapet wall.

* The erroneous statement has been widely published that the crest line of the Canadian Fall has been shortened to the extent of several hundred feet by the power companies for their own benefit. The facts are as stated above. All the excavated material deposited within the Park limits was placed in accordance with the specific directions of the Park Commissioners. No power company has been affected, beneficially or otherwise, by the filling in of the portion of the river-bed from which the water had receded years before the power development began.



SECTIONS OF POWER-HOUSE AND WHEEL-PIT
SHOWING MACHINERY INSTALLED
CANADIAN NIAGARA POWER COMPANY

The dimensions of the building and its relation to the wheel-pit and machinery were determined with great care. The dismantling of the large units, the conveyance of the parts to and from cars, and their temporary storage until the machines were re-assembled, were matters of no small moment, and diagrams were prepared to show the clearance lines of the larger parts when suspended from the two 50-ton cranes with which the building is equipped. The floor in certain sections of the power-house was designed to support the heaviest parts of the machinery arranged in that manner which, of all practical ones, would give the most severe stresses in the supporting beams, the portions of the floor thus designed being marked by brass plugs.

The roof, crane girders, and head-gate girders are carried on steel columns, encased in brickwork, projecting as pilasters into the power-room. The walls are of massive character, suitable for such an installation, being from 26 to 30 in. thick. The outside face is of Queenston blue limestone. This is one of the best limestones for building purposes, and has shown its excellent weathering qualities in the severe climate of Niagara Falls. The position of the power-house is peculiarly trying, as the spray from the Falls often freezes in a thick coating over its entire surface.

The roof is covered with semi-glazed green tile laid directly on steel purlins. It is believed that such a form of construction is somewhat novel for a building of such importance, and where the consequences of leakage would be so serious. The results have been entirely satisfactory, no leakage or condensation taking place since construction ceased. The broad, overhanging, copper cornice combined with the shaded green of the roof gives a most pleasing effect. Figs. 1 and 2, Plate LXXVIII, show the building as now completed. Since the photograph shown in Fig. 1, Plate LXXVIII, was taken, the seal of the Company, designed by Macmonnies, has been cut in the stone over the front entrance. Bronze doors of ornamental design provide access to the power-house and to the visitors' gallery shown on Plate LXXX. Beneath this gallery is an office for the operating staff.

As shown on Plate LXXX, the space below the power-house floor is completely utilized. A subway, 5 ft. in width between the wheel-pit walls and the west foundation wall, provides for piping and auxiliary wiring and cables, and the main subway on the opposite side of the wheel-pit contains the bus-bar structure and the dynamos and feeder

cables. On the east side of the subway, chambers between the inlets contain lockers for the employees, heating and lighting transformers, oil-storage tanks, etc.

The power-house is provided with two 50-ton electric traveling cranes, and these are used together for handling the armature of the generator, this piece weighing about 90 tons. A 5-ton, 3-motor, electric traveling crane in the forebay is used principally for moving racks and stop-logs.

A hydraulic passenger elevator furnishes access from the power-house to the several decks of the wheel-pit.

As shown by Fig. 2, Plate LXXIX, a structure, 127 ft. 6 in. by 17 ft. 6 in. by 11 ft. 7½ in., is built on the power-house floor to enclose the oil switches and to provide a foundation for the switch-boards as well as a working platform for the electricians-in-charge. This enclosure is of ornamental design, and is faced with white tile. Ducts are placed in the walls for carrying the wiring to the switches. Ducts beneath the power-house floor provide for carrying all cables between the generators and the main subway, as well as between the several subways and chambers. Pipes in the pilasters of the building carry wiring to the lights, so that all cables and wiring are concealed.

The view from the roof of the forebay is a beautiful one, the complete panorama of the upper rapids, the Horseshoe Falls, and the parks on both sides of the river being spread before the observer. It is doubtful whether, in any other locality, utility and beauty are combined in such a manner as in the power-plants at Niagara, or where great resources are developed without marring the natural scenery.

Hydraulic Machinery.

The size and gradient of the tunnel having been determined, thus fixing its elevation at the point of juncture with the wheel-pit, the elevation of the working floor beneath the turbines, called the "Rack Deck," was selected so as to keep it above the surface of the tail-race water under all ordinary hydraulic conditions. The carrying capacity of the tunnel is calculated by using 0.01313 as the coefficient of roughness in Kutter's formula, and due allowance is made for the rise of water in the wheel-pit necessary to obtain the velocity maintained by the tunnel. At an early stage it was determined that draft-tubes were to be used, and, in order to keep their outlets submerged

MICHIGAN CENTRAL R. R. MAIN LINE
ERIE & NIAGARA BRANCH MICHIGAN CENTRAL R. R.
Falls View Station
Terminal Developments Co. Tramway Station
Roadway
Conduit
Penstock of Ontario Power Co.
Tunnel
N. P. P. & P. Co. Power House
Forebay
Canal
Bridge
Pumping Station
Tunnel
Well
Tunnel Portal
Falls
Canal
Conduit
Intake
Forebay
N I A G A R A R I V E R

when only a small number of units were in operation, the controlling gate shown on Plate LXXXII was installed. This gate is operated by an 85-h. p. motor, a duplicate motor being provided. The installation and correct alignment of a gate of this size, 23 ft. by 20 ft. 5 in., with a travel of 25 ft., at the bottom of an excavation 171 ft. deep, was quite difficult, but the gate has operated successfully. Indicators are provided to show both the height of the gate and the depth of the water. Although the gate is used daily in the operation of the plant, it has been found that the form and size of the draft-tubes are such that the vacuum is not broken if the outlets are uncovered, and, although the vacuum is not as steady as at other times, no interference with the usual operation would occur if the gate were temporarily inoperative. More particularly during the early operation of the plant, the water passed beneath the gate at a very high velocity, but at the last inspection it was found that no abrasion had occurred, although the vitrified brick invert was worn as smooth as glass.

The general form and arrangement both of the main and auxiliary machinery are shown on Plate LXXX. The water, after passing through the racks, enters masonry inlets, 18 ft. wide, through which it flows to the penstock mouthpieces. Descending into the wheel-pit through the penstock, the water passes into the wheel-case and thence through the twin wheels to a common central draft-chest from which it reaches the tail-race through draft-tubes built into each of the wheel-pit walls.

The total available head, under the usual conditions, will be 136 ft., but, with low water and peak loads, it may be reduced to 130.5 ft., and in preparing specifications for a sixth unit the latter figure was used.

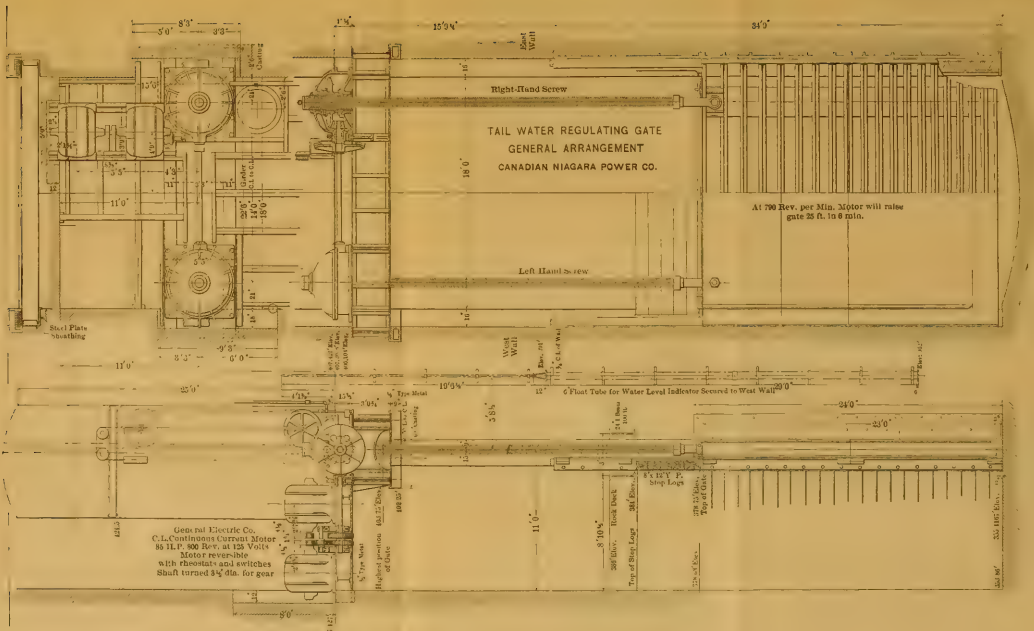
Careful consideration was given to the question of the speed of the units, and, for both hydraulic and electrical reasons, the machines were designed for 250 rev. per min. As the plant was to operate in conjunction with that of The Niagara Falls Power Company, it was necessary to furnish electric current of 25 cycles. The two speeds considered, therefore, were 250 and 188 rev. per min. The latter speed would have increased the weight of the revolving parts and added to the cost of the generator. Although it would have had the advantage of permitting the use of a single wheel, it had disadvantages from the standpoint of the hydraulic design. Experience has confirmed the wisdom of the selection of 250 rev. per min.

The penstocks were made 10 ft. 2 in. in diameter, the maximum size which could be transported on cars to the site of the plant, with flanges left for attachment in the field. The resulting velocity will be 11.2 ft. per sec. when 10 000 h. p. is being developed under minimum head. The entrance to the penstock is bell-shaped, and its axis is inclined at an angle of 11° 7′ with the horizontal. It is composed of cast-iron segments, and an idea of its size is given by Fig. 1, Plate LXXIX, which shows it in place before the end wall of the inlet was built.

The lower penstock elbows are interesting as examples of large steel castings, the over-all dimensions of the complete elbow being 12 ft. 11 in. by 13 ft. 6⅝ in. by 16 ft. 6 in. In order to transport the elbow, it was necessary to make it in two parts, as shown on Plate LXXX. The total weight of the complete elbow is more than 90 000 lb. These castings, for all five units, were made at the Skoda Works, in Pilsen, and, by their smoothness and accuracy of dimensions, illustrate the perfection there attained in work of this character. It may be interesting to note that the test pieces prepared from coupons attached to these castings showed as a maximum the following physical characteristics:

Elastic limit, in pounds per square inch	49 200
Ultimate tensile strength, in pounds per square inch	83 600
Elongation in 2 in	29.8%
Contraction of area	59.0%

Considerable thought was given to the design of the draft-tubes, as the conditions affecting their installation were peculiar. As finally made, they are uniformly 5 ft. 3 in. diameter, and when the unit is running under full load the velocity therein is about 21 ft. per sec. While this high velocity represents a material loss, not only in velocity head, but also in friction, it is useful in imparting to the discharge water a part of the velocity required in the tunnel, the lower section of the draft-tubes discharging at an angle of 45° with the axis of the wheel-pit. To have made the cast-iron draft-tubes sufficiently large to reduce the velocity to the usual limits would have increased the expense greatly, and the conditions are not favorable for the use of concrete-lined draft-tubes. Plate LXXX shows the form of the tubes and the method of supporting the wheel-case on


TAIL WATER REGULATING GATE
GENERAL ARRANGEMENT
CANADIAN NIAGARA POWER CO.
Right-Hand Screw
Left Hand Screw
At 790 Rev. per Min. Motor will raise gate 25 ft. in 6 min.
Steel Plate Sheathing
West Wall
6" Float Tube for Water Level Indicator Secured to West Wall
General Electric Co.
C.L. Continuous Current Motor
85 H.P. 900 Rev. at 125 Volts
Motor reversible
with rheostats and switches
Shaft turned 3¼" dia. for gear
Highest position of Gate
Rock Bank
Top of Slop Logs
Top of Gate

the section immediately below the upper elbow, and Fig. 1, Plate LXXXVI, gives an idea of the appearance of the lower elbow in the wheel-pit before bricking in.

Plate LXXXIII, a cross-section, and Plate LXXXIV, a longitudinal section, show the general arrangement of the several parts. It will be seen that two Francis turbines are used, the water passing from the steel-plate wheel-case to a common cast-iron draft-chest. The regulation is effected by cylinder gates operated by a system of racks and gears actuated by a rod from the governor which passes down the pit along the penstock. The efficiency at partial gate openings might be greater with a regulation by movable vanes in the distributors, yet the practical difficulties of their operation in the space available would be great. Experience has shown that the accurate fit of the cylinder valves between the distributors and the turbine and their accurate guidance is a matter of the greatest importance. Clearances at these points cause vibrations which are most injurious.

The wheel-case and penstock elbow were adjusted to their exact elevation by jack-bolts attached to the supporting castings, and were secured in place by type metal. Space was also provided for horizontal adjustment, and blocks of type metal are used between the vertical faces of the wheel-case base and the corresponding surfaces on the draft-tube brackets. On Plate LXXXIII it will be seen that a space of 2 in. was left between the vertical faces of the flanges on the draft-chest and on the draft-tube, and the draft-tubes were set accordingly, Before Units Nos. 4 and 5 were set, this space was much reduced by the inward movement of the wheel-pit walls, and slip-joints were provided for all the units at these points so that while the vacuum is maintained the flanges are free to move.

The turbine shaft, like all other portions of the solid shafting, is made from oil-tempered nickel steel having an elastic limit of 65 000 lb. per sq. in. It is guided by three bearings, two above the upper runner and one below the lower runner, the shaft being divided into two parts for convenience in dismantling. Difficult features of the design are: to prevent the escape of the oil used for lubricating these bearings, and to keep water out of the oil. Though not thus shown on Plate LXXXIII, this is done by the use of cup-leather packings, it having been found that hemp packings scored the shafts.

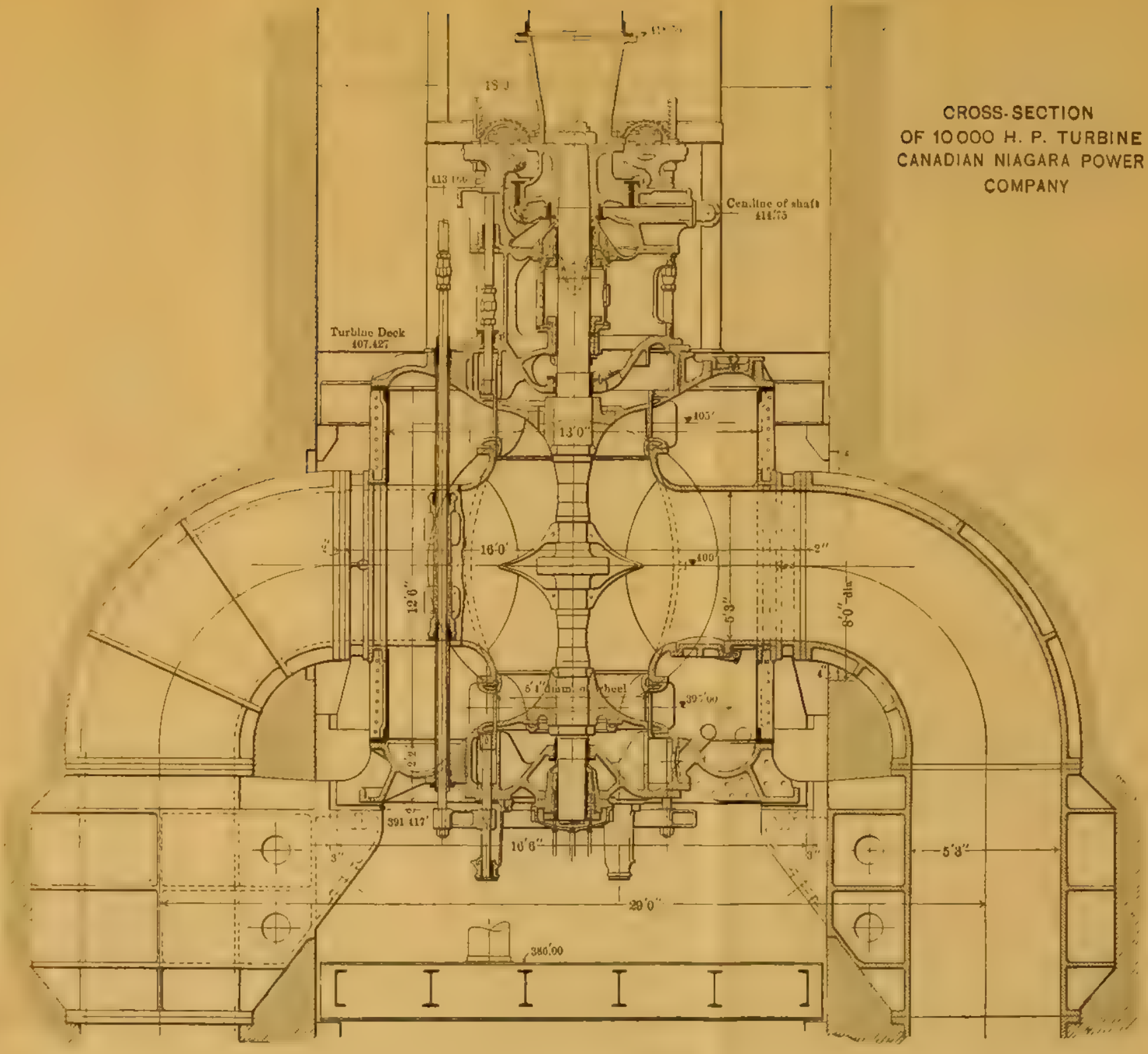
The runners and distributors are of bronze which, for Units Nos. 4 and 5, shows the following physical and chemical properties:

Elastic limit, in pounds per square inch	15 500
Ultimate tensile strength, in pounds per square inch	31 400
Elongation in 2 in	25.6%
Copper	88 %
Tin	10 %
Zinc	2 %

The bronze for the runners on Units Nos. 1, 2, and 3 contained some lead, and a recent specification by the writer requres 85% of copper, 10% of tin, 3% of lead, and 2% of zinc.

Special interest attaches to the method of supporting the revolving parts of the hydraulic and electrical machinery, weighing 240 000 lb., moving at the rate of 250 rev. per min. By reference to Plate LXXXIII, it will be seen that by means of an opening in the wheel-case base ring, surmounted by an elbow, water passes to the space below the lower runner, which is thus under the full wheel-case pressure. The space above the upper runner communicates freely with the draft-chest through the openings shown on Plates LXXXIII and LXXXIV, and pressure, therefore, cannot accumulate there from the water which leaks past the upper circular valve. With this arrangement, the upward pressure beneath the lower runner is not balanced by a pressure above the upper runner. Openings were provided for supplying the chamber beneath the lower runner with water from an outside source, but they have not been found necessary. The upward pressure thus provided for is not sufficient to support the entire rotating weight, and additional balancing pressure is effected by the piston shown immediately above the upper bearing on Plates LXXXIII and LXXXIV. A 12-in. pipe is carried down the wheel-pit to connect by two 6-in. pipes with opposite sides of the space beneath the piston, an emergency connection being made with the wheel-case. The outside diameter of the piston being 3 ft. 10 in. and the inside diameter 21½ in., there is thus an area of 1 298 sq. in. subjected to the pressure of the water, which is about 43 lb. per sq. in. The water which leaks out at the inner and outer circumferences—where a clearance of $\frac{1}{32}$ in. is provided—is carried away by the drain pipe shown on Plate LXXXIV. Valves provide means of regulating

CROSS-SECTION
OF 10 000 H. P. TURBINE
CANADIAN NIAGARA POWER
COMPANY



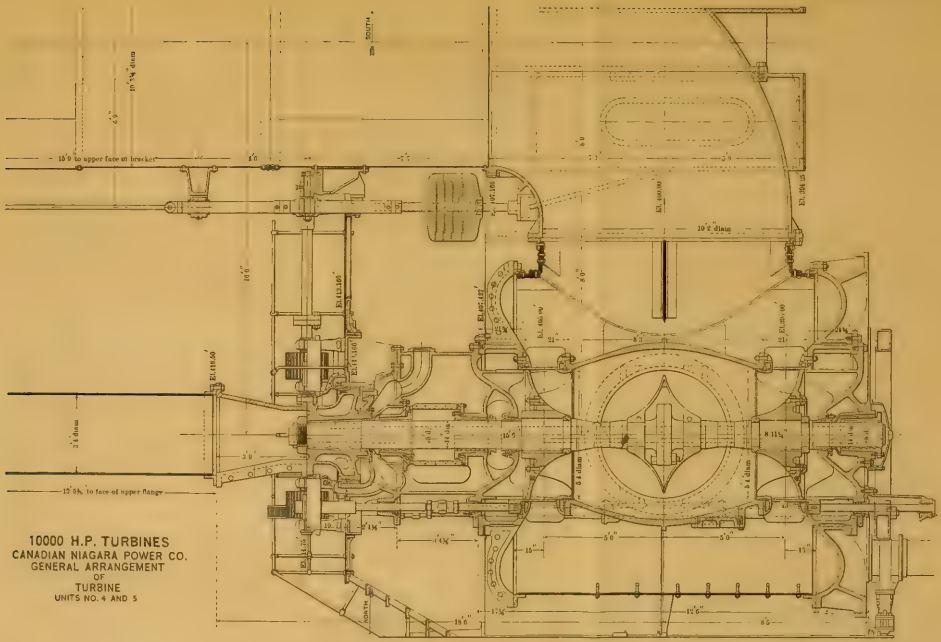
the pressure under the piston. Bronze sleeves and bushings are placed on the moving and stationary parts at the periphery of the piston.

It would be possible, of course, to design the balancing piston so that, in conjunction with the pressure under the lower runner, it would exactly balance the weight of the revolving parts under a given condition of load on the unit and elevation of tail-water; but such conditions are changing, and provision must be made, not only to take care of their variations, but also to provide for failure of the water pressure under either the balancing piston or the lower wheel, or under both. This has been done by the installation of an oil-step bearing, placed at the deck immediately below the power-house floor, and supported by the heavy girders shown on Plate LXXX. It consists of two disks of charcoal iron, one stationary and the other secured to the shaft, oil being forced between these disks by a special pump driven from the main shaft. As much interest has been manifested in these bearings, by visitors at the plant, they are shown on Plate LXXXVII. It will be seen that a guide-bearing is placed just below the step-bearing, the oil-catcher and up-thrust ring being shown at the bottom of Plate LXXXVII. The stationary disk is attached by dowels to a receptacle having a spherical seat, the latter being provided to compensate for any errors in alignment and for adjustment to obtain even distribution of oil. The nut on the shaft permits the accurate setting of the upper disk. The outer cover protects the several parts, and acts as a receptacle for the oil. The cast-iron base is set with the upper finished surface at the proper elevation, and type metal is then poured between it and the top of the girders. Adjusting screws permit the correct alignment of the bearing, after which it is doweled to its final position. The bearing disks are made in halves, scraped together, and united by two bolts on each side, the joint preferably being tongued. The disks are made from the best quality of iron obtainable, the tensile strength being as high as 34 000 lb. The bearing surfaces contain two annular grooves, the outer one $1\frac{1}{2}$ in. wide and the inner one 1 in. wide. They are connected by radial grooves, and the surface is still further subdivided by other grooves making an angle of 45° therewith. As will be seen on Plate LXXXVII, oil is admitted to two cored channels extending about 100° on opposite sides of the bearing, being fed therefrom through three openings to the inner annular space. The oil is carried to all parts of the bearing,

and escapes at both the inside and outside periphery. As the pump supplying the oil is of the three-plunger pattern, moving at constant speed, the bearing acts automatically, the pressure of the oil corresponding to the weight to be lifted. These bearings have never been called upon to carry the entire rotating weight, but it is believed that they would do so successfully. Unit No. 1 was operated for some time with the balancing piston removed, and Units Nos. 4 and 5 were each run for 1 hour with the balancing piston pressure cut down to 5 lb., the rise in the temperature of the oil being only 4°, 1 gal. of fresh oil per minute being admitted to the suction tank of the pump. The pressure of the oil rose to only 110 lb. per sq. in., although the pumps are capable of exerting a pressure of 500 lb. per sq. in. In the writer's opinion, the success or failure of an oil-step bearing depends fully as much on the workmanship as on the design. In the bearings now being described the surfaces of the disks were finished and the grooves cut; then they were ground and scraped, and finally the surfaces were ground together with flour of emery, followed by chalk. It is also very important that the backs of the disks be scraped to an accurate bearing on their receptacles.

An interesting question arose as to the maximum capacity of a prime mover, of the general design of the present units, under the conditions imposed by the existing construction of penstock elbows and draft-tubes. The total loss of head in the unit, under various load conditions, had been determined by the Pitot-tube method, yet, in order to solve the new problem, it was necessary that the total head lost should be differentiated and the loss determined in each section through which the water passes. Owing to the turns in the penstock and draft-tube, it was difficult to do this theoretically, and, therefore, on October 8th, 1907, an experiment was made in the following manner:

Pipes were connected with the penstock radially at four axial points just below the slip-joint near the upper elbow. Similar connections were made immediately above the lower elbow. The pipes from each elevation were united and carried to the upper wheel-pit deck where they were attached to glass tubes on gauge-boards. A pipe was also tapped into a connection with the forebay and another was connected with the gauge pipe attached to the wheel-case opposite the penstock elbow. The gauge-boards for these pipes were placed near those first



10000 H.P. TURBINES
CANADIAN NIAGARA POWER CO.
GENERAL ARRANGEMENT
OF
TURBINE
UNITS NO. 4 AND 5

mentioned so that the four gauges could be read simultaneously. One of the draft-tubes was tapped at four points on the periphery, and the vacuum read on a mercury gauge. As the elevation of the water in the tubes surged rapidly, readings were taken every 5 sec. simultaneously on the several gauges, fifteen tests of 5 min. each being made with various loads on the generator. Six tests were made at practically full load, and the following shows the average distribution of the losses from the average total head of 134.56 ft. between head-water and tail-water:

From forebay to bottom of upper elbow	8.33%
In straight vertical portion of penstock	0.34%
In lower elbow and wheel-case	11.83%
Friction loss in draft-tubes	11.30%
$\frac{V^2}{2g}$ in discharge water	18.95%
In turbine wheel, and mechanical losses	49.25%
	100.00%

Curves were plotted to show the losses as segregated, and these were prolonged to determine the increased losses which would result from greater discharges. It was then determined that, with turbine machinery of the present design, in conjunction with existing draft-tubes and penstock mouthpieces, the maximum available power per unit on the turbine shaft would be about 11 500 h. p., and that beyond that point the losses would more than counterbalance the power contained in increased quantities of water. This, however, should not be regarded as a criticism on the present design, as it performs efficiently the work for which it was intended.

Piping Systems and Auxiliary Machinery.

The lubricating and high-pressure oil systems are of complicated design, and only their more important features will be described.

Brass piping and bronze fittings are used throughout for the passage of oil, because of the increased carrying capacity and also the reduced chance of scale being carried by the oil into the bearings.

Lubrication is required for seven guide-bearings on each main shaft in addition to that necessary for the exciters. High-pressure oil

is furnished to the step-bearing and governor of each unit by separate pumps. This is a departure from the method used in the oiling system in Power-House No. 2 of The Niagara Falls Power Company, upon which are modeled many parts of the lubricating system which is being described. In the American plant the same pumps supply both the lubricating oil and the high-pressure oil for the governors, but such an arrangement necessitated the installation of accumulators and special reducing valves.

For convenience in operation, and to avoid the placing of apparatus below the level of the water in the tail-race, the lubricating system is divided into upper and lower systems, the latter supplying the turbine bearings and the exciters, and the former the oil required by the remaining bearings. The oil-cleansing apparatus and pumps are in a brick-lined chamber cut in the rock on the east side of the wheel-pit near its middle. The chamber is in three stories, the main portion, 50 ft. 4 in. by 17 ft. 5 in., being at the level of the turbine deck, while the front of the chamber is carried to a depth of 24 ft. below that level in order to receive the drainage from the lower bearings.

Oil is brought into the power-house in a tank car, and is piped directly to storage tanks in a chamber beneath the power-house floor. From these tanks it is piped to the oil chamber at the turbine deck, fresh oil being admitted to the system as required. From the chamber the oil from the upper system is pumped to equalization tanks in the power-house roof, from which it passes to a main laid just above the thrust deck. With this main are connected at each unit two pipes, one leading to the generator bearings and the other to the remaining guide-bearings of the upper system. Connections are provided for introducing fresh oil into the high-pressure system. Oil having passed through a given bearing is thrown into the oil-catcher and is piped to a funnel where its temperature may be observed. A drain pipe common to the discharge from the several bearings is provided at each unit, and leads to a horizontal pipe above the turbine deck, through which the oil is conducted to its chamber. Here it may be treated in one of three ways: 1, it may pass directly to suction tanks and thence be pumped by Quimby screw pumps to the equalization tanks in the power-house roof; 2, the oil may be filtered through cotton cloths in a given compartment of a four-chamber tank, heat being used as desired to assist in the deposit of the foreign matter;

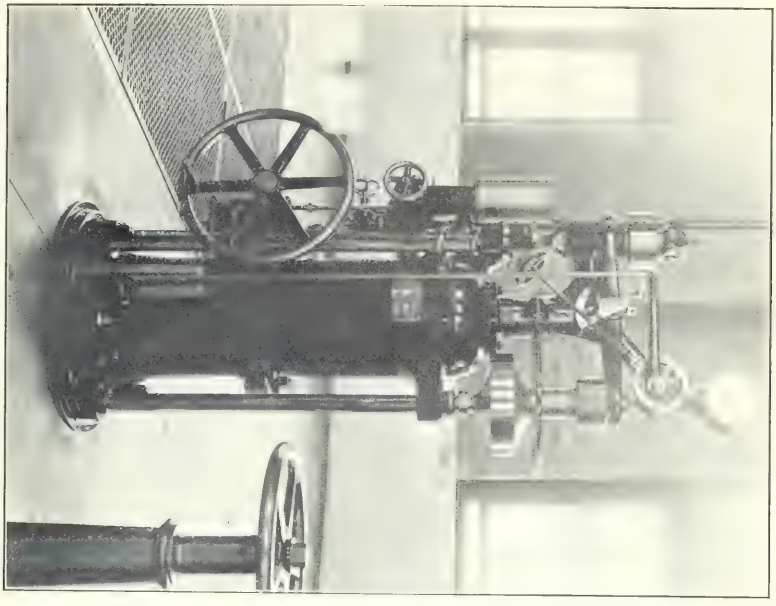


FIG. 1. OIL-PRESSURE SINGLE-ACTING GOVERNOR.

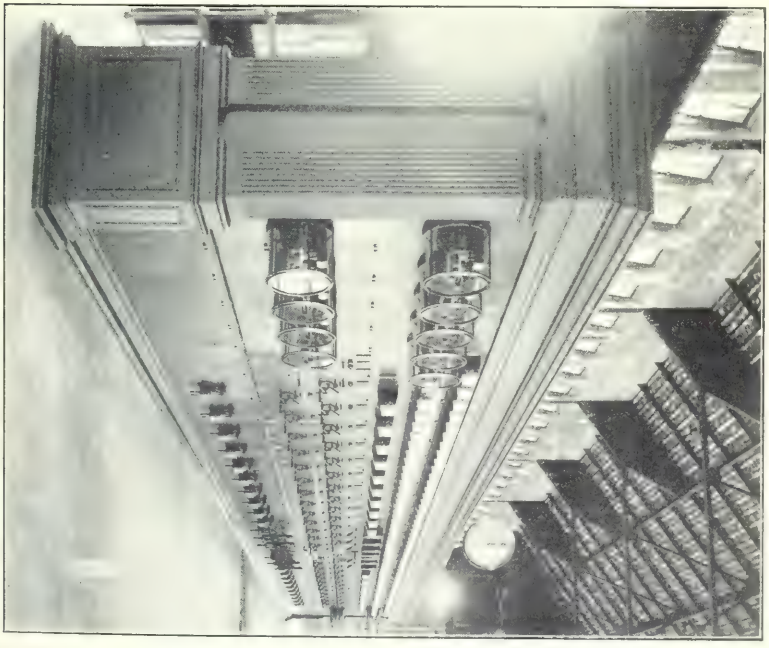


FIG. 2. SWITCH-BOARD.

3, the oil may be boiled to remove any excess of moisture, the oil being drained to cooling tanks before being filtered or pumped direct to the upper tanks.

The oil in the lower system receives a certain quantity of water from the lower turbine bearing, and therefore requires a different treatment from that of the upper system. The return oil enters one of two suction tanks, and is pumped by a motor-driven Quimby pump to a separating tank in the upper story of the chamber. This tank is provided with baffle-plates, which cause the oil to pass in a circuitous course, moderate heat being applied. If necessary to boil the oil to remove the residual water, it passes to the tank used also for the upper-system oil, thence to cooling tanks, from which it flows to a filter tank similar to that used for the upper system; thence it goes to the suction tank and is pumped to a horizontal supply main placed above the turbine deck and provided with a stand-pipe and overflow. By-passes are inserted, so that any of the purifying processes may be omitted when unnecessary. All apparatus is in duplicate, so that no interruption of the service may occur through failure of oil.

The high-pressure oil for the governor and step-bearing of each unit is supplied by a pair of triplex pumps, each having a capacity of 100 liters, driven from the main shaft and capable of supplying a pressure of 500 lb. per sq. in. The working pressure of the oil supplied to the step-bearing varies automatically with the load to be carried, and the pressure of the oil pumped to the governor is normally 225 lb. per sq. in. Complete cross-connections and relief valves permit either pump to be used on the governor or step-bearing at will, and a main above the thrust deck enables a spare motor-driven pump of the same style to supply high-pressure oil to any unit when it is being started. If any accident should disable the motor-driven pump, oil from the pumps of a moving unit will supply one which is idle.

The governors for the main units are of the oil-pressure, single-acting type, shown in Fig. 1, Plate LXXXV, and are patented by Escher Wyss and Company. A compression member connected with the piston in the main cylinder extends to one end of a lever on the thrust deck, and, from the opposite end of the lever, a rod, as already explained, passes to the rack and pinions operating the cylinder gates, a weight of about 7 000 lb. being attached to the lower end of the rod. The gates close by gravity and open by the oil pressure. As showing

the accuracy of regulation which obtains, the following extract from the contract with the manufacturer may be of interest. It requires:

"(2)—That each and every governor to be manufactured hereunder shall give the following regulation, namely: When sudden changes in the load on the generator corresponding to the respective percentages of the total rated capacity of the generator which are set forth in the first column of the following schedule are made, the respective variations in speed above or below the normal shall not exceed the respective percentages of the said normal speed which are set forth in the second column of the said schedule, and the times which will elapse between the respective changes of load and the restoration of normal speed shall not exceed those respectively set forth in the third column of the said schedule."

" Changes in Load.	" Corresponding Maximum Speed Variation.	" Time of Perturbations. in Seconds.
25%	2%	25
50%	4%	35
75%	6%	50
100%	8%	60

In testing the governor, the above percentages of load were thrown off with an oil-switch, thus making the tests as severe as possible. The regulation required when throwing off load was practically fulfilled, the regulation with the removal of partial loads being better than that required, while the increase in speed when the full load was thrown off was about 10% instead of 8 per cent. Greater perturbations in speed than those specified in the contract were caused by the sudden addition of loads, especially of large increments. This condition was due to the lack of oil capacity rather than to the governor action, and air-pressure chambers, supplied by a separate motor-driven air compressor, were installed on the oil piping, but have not been used, as, under the operating conditions of the plant, the instantaneous additions of large increments of load are most unusual. The regulation obtained has proven very satisfactory under operating conditions.

In the northerly chamber, at the level of the turbine deck, are installed three exciter units, each consisting of a 200-kw., 125-volt, vertical-shaft, direct-current dynamo, driven at 600 rev. per min. by a turbine of the Francis type. The direct current thus generated is used, not only for exciting purposes, but also for operating motors and for furnishing light and heat. The arrangement with vertical shaft-

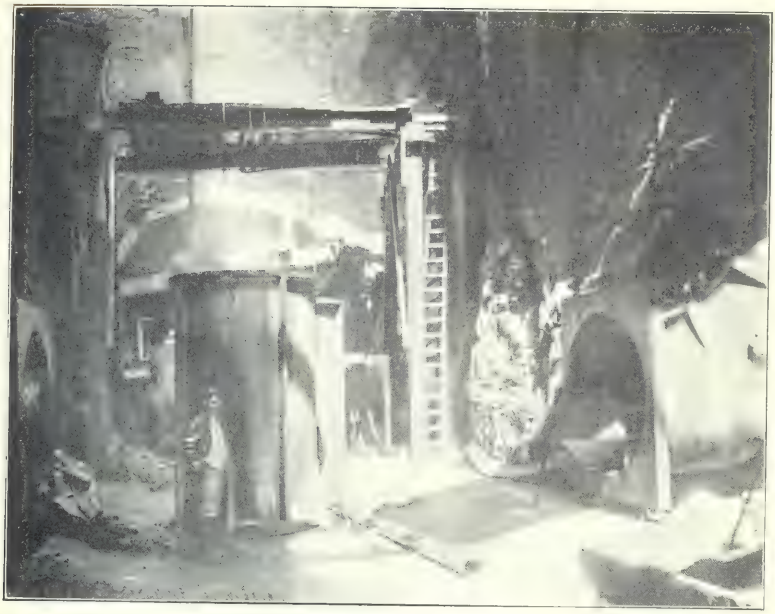


FIG. 1.—PLACING DRAFT-TUBES

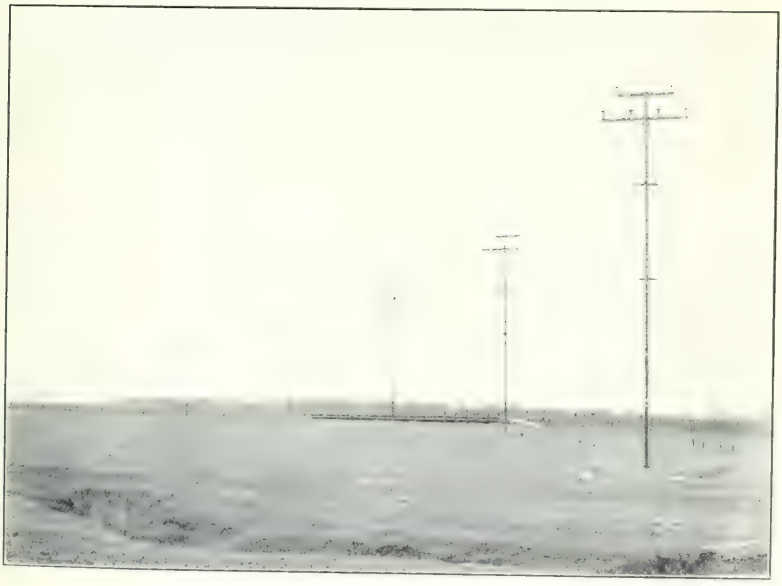


FIG. 2.—TRANSMISSION LINE.

ing permits the electrical machinery, switch-boards and wiring to be located on the upper floor of the chamber, removed from the hydraulic machinery and piping, which are installed on the floor below, the draft-tubes occupying the space under the turbine floor. Except as noted below, all auxiliary machinery is supplied with water for power purposes from a 36-in. riveted-steel pipe connected with the north end of the forebay and laid along the rack deck. Branch pipes, provided with control valves, lead to the several turbines. Each exciter turbine has a separate supply pipe, thus tending to prevent a total shut-down. The 36-in. pipe is fed at various points by 24-in. pipes connected with the forebay.

As the continuity of the long-distance service is practically as dependent upon an uninterrupted supply of water to the transformers as upon the electric current, and as no reliable municipal source of water was available, it became necessary for the company to construct its own water-works system. Accordingly, there have been installed, in Chamber No. 2, two 3 000 000-gal., single-lift, centrifugal pumps, each of which is driven by a 174-h. p. Francis turbine. One of the pumps is also connected with a motor. As it is so important to provide against a shut-down, both suction and motor pipes are provided in duplicate. In addition to connections to the 36 and 24-in. pipes above referred to, pipes are also connected with the penstock on Unit No. 3, so that if, under any circumstances, the smaller pipes should become choked with ice, the penstock will be available as a source of supply. Provision is made for two additional pumps if they should be required. The pumps normally operate under a pressure of 137 lb. per sq. in., but, by connecting them up in series, as may be done quickly in case of fire, the pressure can be raised to 225 lb. per sq. in. A 24-in. main carries the water to a point outside of the power-house where a system of pipes leads in various directions, the most important main leading to the transformer station and stand-pipe on the company's lands at the top of the escarpment. The stand-pipe, 30 ft. in diameter and 115 ft. high, is on a branch line in which is placed a motor-operated valve to be closed from the transformer station in case of fire. Provision is made for supplying with water the extensive factory sites owned by the company between the transformer station and Chippawa. A 4-in. main is secured to the wall above the thrust deck, and, at each unit, connections are taken off to

the cooling coils in the generators and to hose outlets at the various wheel-pit decks and the power-house floor. Ample provision is thus made for extinguishing fire, although such precautions would seem to be in the nature of making assurance doubly sure, as the power-house and wheel-pit decks are absolutely fire-proof, practically the only wood being that in the southerly temporary end of the power-house, upon which a sprinkler system is installed.

In addition to the oil-treating apparatus already described, there is installed in Chamber No. 3 an air compressor operating at 140 lb. per sq. in., driven by a 7-ft. impulse water-wheel, the compressed air being used for cleaning out the generators and for operating various tools.

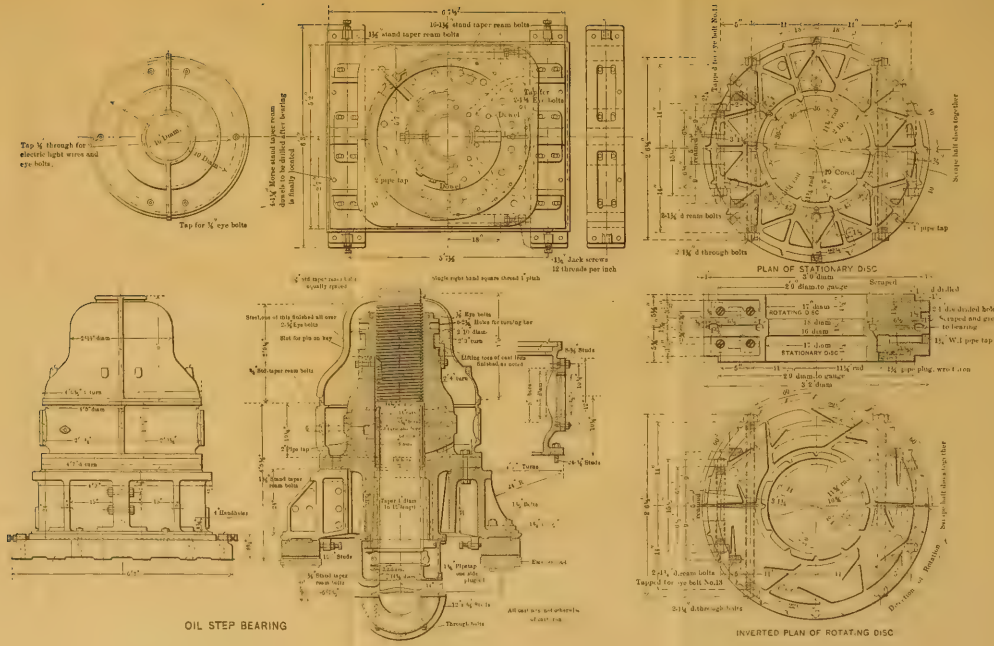
Electrical Installation.

Only a brief description will be given of the more important features of the electrical installation. For a more detailed account of this part of the plant, reference is made to an article* by Mr. H. W. Buck, who, as Electrical Engineer of the Canadian Niagara Power Company, has been in responsible charge of the design and installation of the electrical plant.

As shown on Plate LXXX, the generators are carried on stone-faced, concrete-filled arches, being secured to cut-stone foundations 6 in. above the power-house floor. The tops of these foundation stones were cut with great exactness after the turbine shaft was in place, as no grouting or other filling was placed between the generator base and the foundation.

The generators are of the revolving, internal-field type, furnishing 25-cycle, three-phase current at 12 000 volts. The machines are rated at a capacity of 10 000 e. h. p., but they are capable of a large over-load. The general construction of the generator is shown on Plate LXXXVII. Exciting current at 125 volts is supplied to the collector rings immediately below the upper shaft coupling. From the generator the current is carried in ducts to the main subway and thence to double-throw selector oil-switches, through which it passes to the four sets of bus-bars. The oil-switches, in addition to their automatic action in case of short circuits, are operated by relay switches located on panels on the floor of the switch-board gallery shown on Fig. 2,

* *Proceedings*, Sixteenth Convention, Canadian Electrical Association.


Tap ¾" through for electric light wires and eye-bolts.
Tap for ⅝" eye bolts
16-1⅜" stand taper ream bolts
1⅜" stand taper ream bolts
Taper for 2¼" Ey. bolts
Dowel
Dowel
1" pipe tap
18"
5'-7⅜"
1⅛" d. jack screws
12 threads per inch
Bore 4 holes for bolt No. 11
Scrape half draw together
19 Coned
8-1⅜" d ream bolts
1" pipe tap
2-1⅜" d through bolts
PLAN OF STATIONARY DISC
2'-9" diam
2'-0" diam. to scrape
Scraped
d drilled
ROTATING DISC
17" diam
15" diam
16" diam
STATIONARY DISC
2-1" dia. dowel holes
Scraped and ground to bearing
1¼" W.I. pipe tap
11¼" rad
2'-9" diam. to scrape
2'-5" diam
¼" pipe plug, wrought iron
Steel Lens of this finished all over
2-¾" Eye bolts
Slot for pin in bar
Rod-taper ream bolts
Lifting lugs of cast iron finished as cored
8-⅞" Studs
4-⅞" Studs
Turns
1½" Bolts
1¼" Pipetap one side plugged
Through bolts
All cast iron not otherwise shown
OIL STEP BEARING
Tapped for eye bolt No. 13
2-1⅜" d. draw bolts
2-1⅜" d through bolts
Scrape half draw together
Direction of Rotation
INVERTED PLAN OF ROTATING DISC

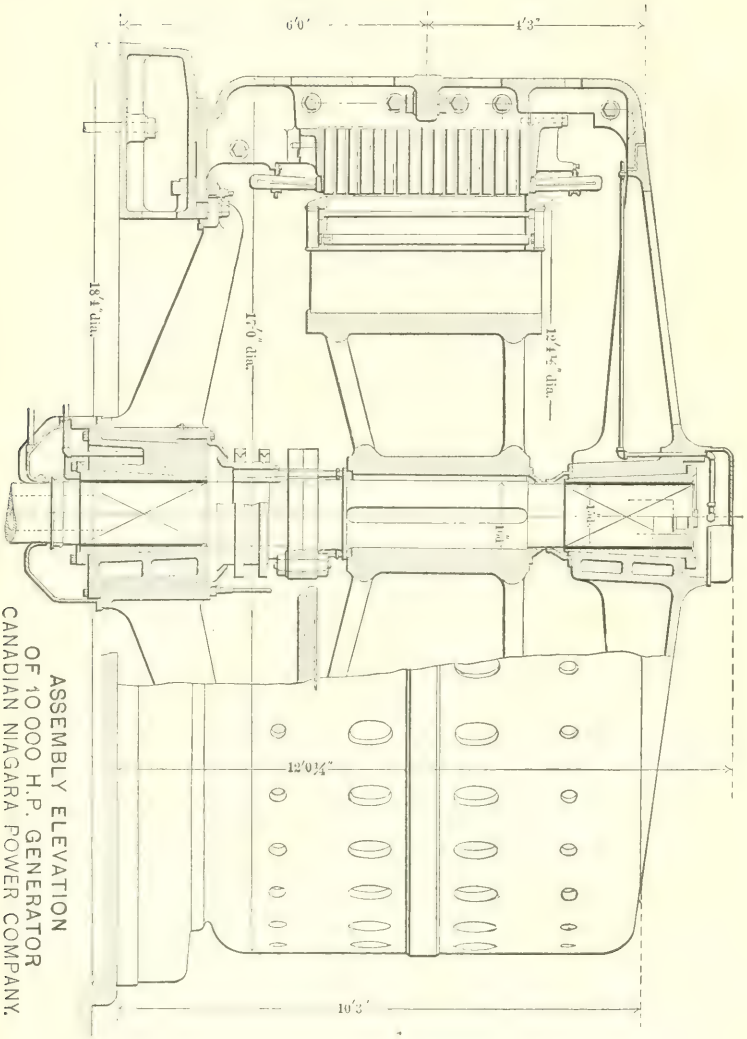


FIG. 4.

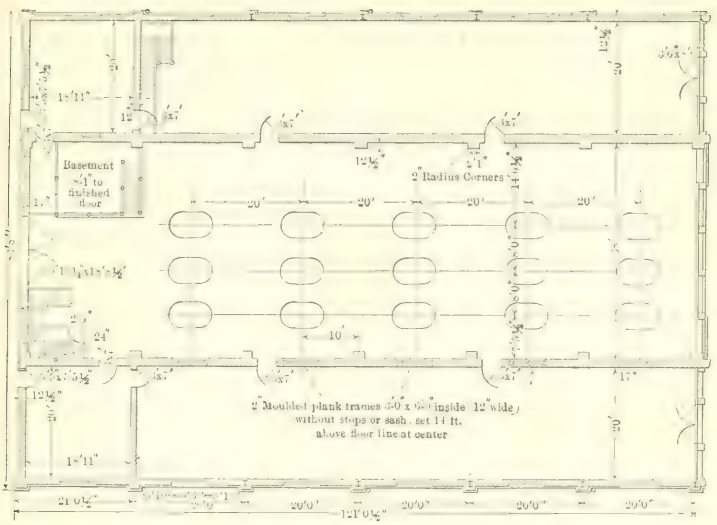
ASSEMBLY ELEVATION
OF 10 000 H.P. GENERATOR
CANADIAN NIAGARA POWER COMPANY.

Plate LXXXV. The operating board contains five generator panels, twenty feeder panels, ten recording wattmeter panels, and three bus-bar interconnecting panels. Each panel is complete in itself, and contains no instruments except those belonging to the apparatus or feeders controlled thereby. The switch-board shown in Fig. 2, Plate LXXXV, is designed to provide for five units, the remaining six units being governed by a second switch-board to be located opposite the eighth unit. The sixth unit, if installed in advance of others, however, will be controlled, at least temporarily, from the existing board. Emergency switches are provided by which the field current may instantly be thrown off the entire number of units should occasion require.

All the cables are carried singly upon grooved electrose supports secured to horizontal channels. In case of a short circuit, it is believed that no arc will form between the cables; and their thorough separation thus minimizes the possibility of trouble resulting from burn-outs. The cables are carried to a connection with the two main conduits, namely, that leading through the Queen Victoria Niagara Falls Park to the upper steel arch bridge across the Niagara River, and that leading to the transformer-house at the top of the escarpment. The former group interconnects the plants of the Canadian Niagara Power Company and The Niagara Falls Power Company. The current from the Canadian plant may be supplied directly to those tenants of the American company who use it at 11 000 volts; it may be raised to a voltage of 22 000 and sent to Buffalo; or, by using Scott connected transformers, it may be reduced to 2 200 volts at two-phase, and the machines in the two plants operated in parallel.

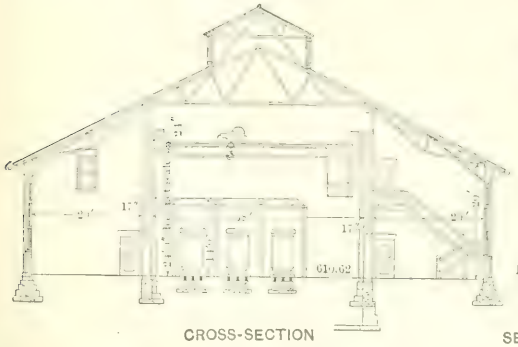
The transformer station, located as shown on Plate LXXXI, is a brick and steel structure, which is divided longitudinally into three sections, as indicated on Fig. 5. The east side contains the low-tension switches and air-cooled 11 000-2 200-volt transformers; the central bay contains the main, oil-insulated, water-cooled transformers; and, in the west compartment, are the high-tension switches and bus-bars. By this arrangement of the building, the several classes of apparatus are completely separated from each other, and, as the building is absolutely fire-proof, continuity of service is secured as well as possible. The main transformers, by which the voltage is raised for overhead transmission to Buffalo, are wound so as to be capable, with

Drummond-ville
NIAGARA FALLS
Transformer Station
Power Houses
GOAT I.
Power House
Transformer Station
NIAGARA
City Line
LaSalle
Town Line
N.F.P.CO. TRANSMISSION LINES
WHEATFIELD
Niagara River
City Line
NO. 142 POLE LINES
NORTH TONAWANDA
POLE LINE TO LOCKPORT
BUCKHORN I.
Burnt Ship I.
NAVY I.
Chippawa Cr.
STAMFORD
Chippawa
Town Line
Usher Cr.
Sub Station
GRAND ISLAND
Tonawanda Creek & Erie Canal
Chippawa Chan.
Erie Canal
POLE LINE NO. 1
TONAWANDA
Line
City
MICHIGAN CENTRAL R.R.
Hayer Cr.
WILLOUGHBY
Black Creek Station
Black Creek
Tonawanda Chan.
POLE LINE TO TERMINAL HOUSE A 22.40 MILES
Erie Canal
POLE LINE NO. 2 TO TERMINAL HOUSE A 29.38 MILES
N.Y.C. & H.R.R.R.
TONAWANDA
Niagara River
Town Line
City Line
CNPP CO. TRANSMISSION LINES LENGTH TO TERMINAL HOUSE 8-15 pc MILES (64982)
Terminal House A
BERTIE
M.C.R.R.
Bridgeburg
N
Niagara Jc.
M.C.R.R.
GD.TR.R.R.
TRUNK RD.
Fort Erie
Terminal House B
BUFFALO
GRAND GOODERICH
Lake Erie
GENERAL MAP OF
TRANSMISSION LINES
FROM
NIAGARA FALLS TO BUFFALO
CANADIAN NIAGARA POWER COMPANY

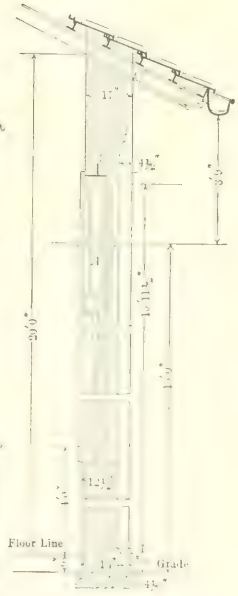


GENERAL FLOOR PLAN

FRONT ELEVATION



CROSS-SECTION



SECTION OF OUTER WALL

MAIN TRANSFORMER STATION

FIG. 5.

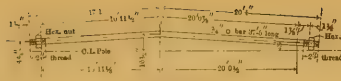
an 11 000-volt primary current, of giving a secondary current of either 22 000, 33 000, 38 000, or 57 300 volts. The first arrangement is now used, as the Canadian plant frequently operates in parallel on the Buffalo load with the plant of The Niagara Falls Power Company. All the wiring, however, is insulated to carry safely the maximum available voltage. The fifteen existing transformers have a total capacity of 25 000 e. h. p., the small size being adopted for flexibility in operation. This end having been attained, the transformers to be installed in the building when extended are planned to be of 4 167 e. h. p. capacity, it being contemplated to double the capacity of the station. In addition to water and drain pipes, provision is made for supplying oil to the transformers and drawing it to an air-tight tank through lead-lined cast-iron pipes. In case of emergency, the oil may be discharged from the transformers into a sewer. Such current as is required locally is transmitted overhead at 2 200 volts, only a small load of this character now being carried.

Conduits.

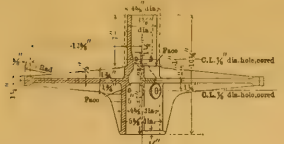
Two main trunk conduits, each containing 32 ducts, were built in connection with the power development of the Canadian Niagara Power Company. These are shown by broken lines on Plate LXXXVIII. The length of the conduit to the arch bridge is 6 500 ft., and of that to the transformer station, 2 200 ft. These conduits are of interest because of the arrangement of the ducts in groups of eight each, it having been found, in the experience of The Niagara Falls Power Company, that injurious heating might be expected if ducts containing cables carrying large amounts of current were grouped in a mass, as has frequently been done elsewhere. The double manhole shown by Fig. 6 has proven very satisfactory. The results of a short circuit are minimized, the cables have a straight lead, with maximum splicing space, and the workmen have abundance of room. The advantages of this design, as compared with the elliptical and hexagonal manholes frequently used, are apparent.

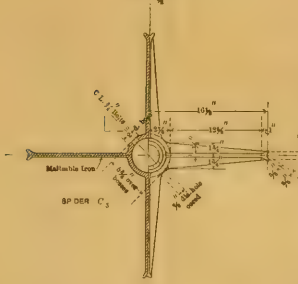
Transmission Lines.

The output of the transformer station is now used principally in supplying 22 000-volt current to The Cataract Power and Conduit Company at a new station built near their load center in Buffalo.

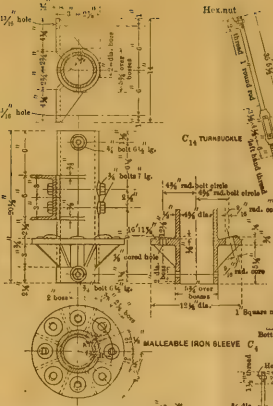


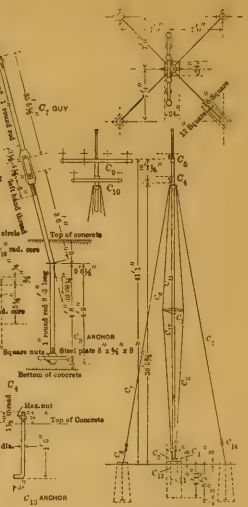
C_6 TRUSS RODS

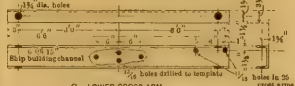




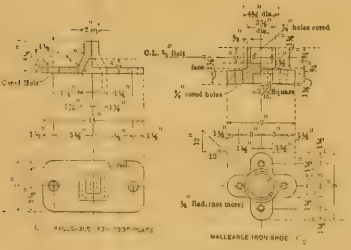
Note.—Holes for ¾″ bolts are to be drilled solid 7/16 dia. through C_7 and C_{11} in same operation except those in the lugs which are to be drilled to template.

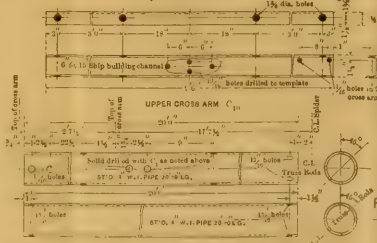






C_9 LOWER CROSS ARM





For 1 Pole	Name	Remarks
1	C_1 Foot plate	
1	C_2 Shoe	
1	C_3 Spider	
1	C_4 Sleeve	
1	C_5 Cap	
6	C_6 Truss rods	
4	C_7 Guy rods	
4	C_8 Anchors	
1	C_9 Cross arm	
1	C_{10} " "	
1	C_{11} Pipe	
1	C_{12} "	
2	C_{13} Anchor bolts	
4	C_{14} Turnbuckles	
4	Bolts (Cross arms) ⅝″ x 2′ 1″ Hex. head and nut.	
4	" (Cross arms) ¾″ x 2′ 8″ " " "	
2	" (Shoe and fly nut) ¾″ x 0′ 5½″ " " "	
3	" (Sleeve and Spider) ¾″ x 0′ 5½″ " " "	

REVISED PLAN AND DETAILS OF STANDARD POLE FOR NUMBER 2 TRANSMISSION LINE TO BUFFALO.

CANADIAN NIAGARA POWER COMPANY.

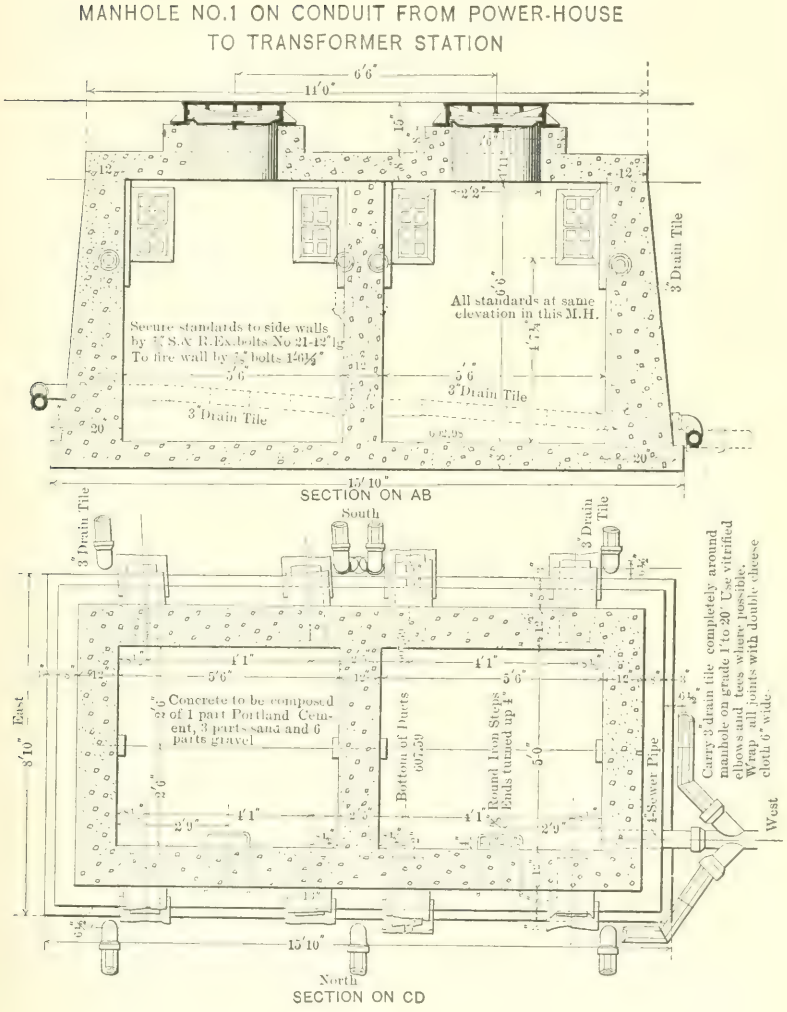
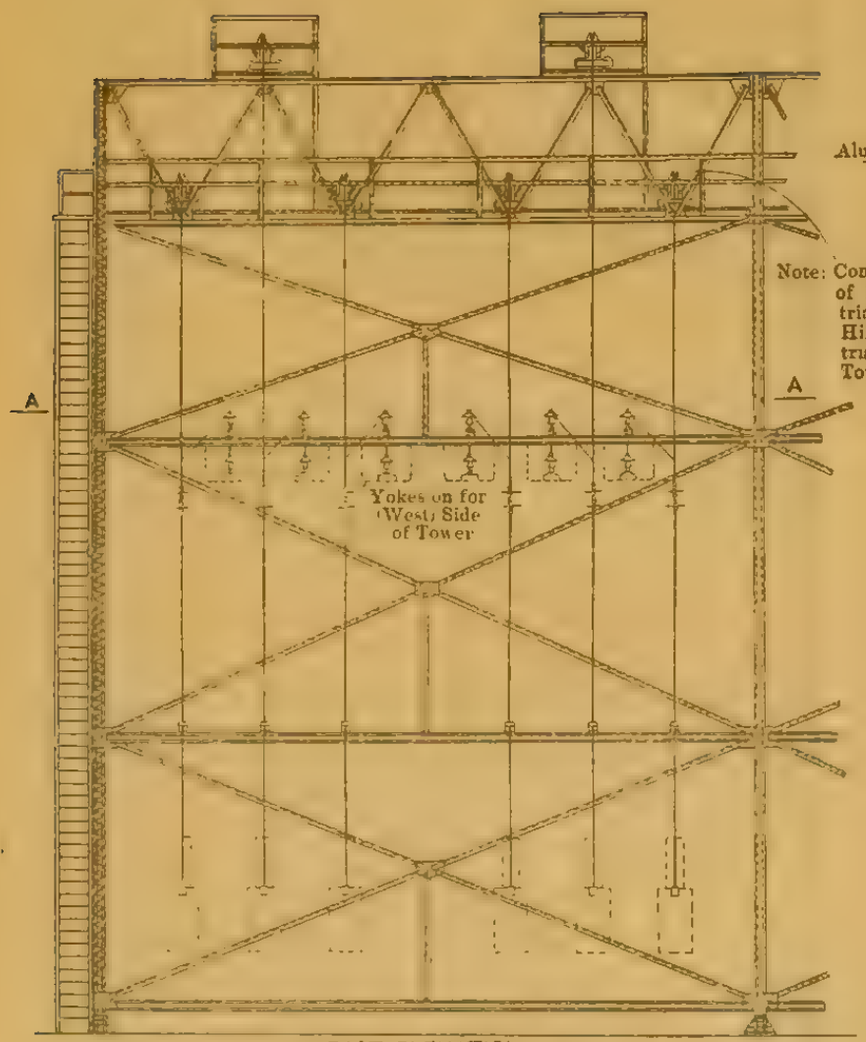

MANHOLE NO.1 ON CONDUIT FROM POWER-HOUSE
TO TRANSFORMER STATION
6'6"
11'0"
12"
15"
2'2"
3" Drain Tile
All standards at same elevation in this M.H.
Secure standards to side walls by 3/4" S.A.E. R.E.A. bolts No 21-12"lg
To fire wall by 3/4" bolts 1 6½"
5'6"
3" Drain Tile
20"
6'2.98
15'10"
SECTION ON AB
South
3" Drain Tile
4'1"
Concrete to be composed of 1 part Portland Cement, 3 parts sand and 6 parts gravel
Bottom of Ducts 60'.50
Round Iron Steps Ends turned up 4"
5'0"
4" Sewer Pipe
Carry 3" drain tile completely around manhole on grade 1 to 20 Use vitrified elbows and tees where possible. Wrap all joints with double cheese cloth 6" wide.
West
8'10" East
2'6"
2'9"
15'10"
North
SECTION ON CD

FIG. 6.

That company was formerly supplied with power by The Niagara Falls Power Company exclusively, but now has the advantage of duplicate service. The Canadian transmission lines are built on a private right of way, 30 ft. in width, following the route shown by Fig. 6 from the transformer station to the Town of Fort Erie, Ontario, and thence by an overhead crossing of the Niagara River to Terminal House B, in Buffalo, a distance of 16 miles, as compared with 25 miles for the American transmission. One pole line containing two circuits is in service, and the building of a second line is now in progress. Each circuit has a rated capacity of 12 500 h. p. or a total transmitting capacity over the right of way of 50 000 h. p. at 22 000 volts.

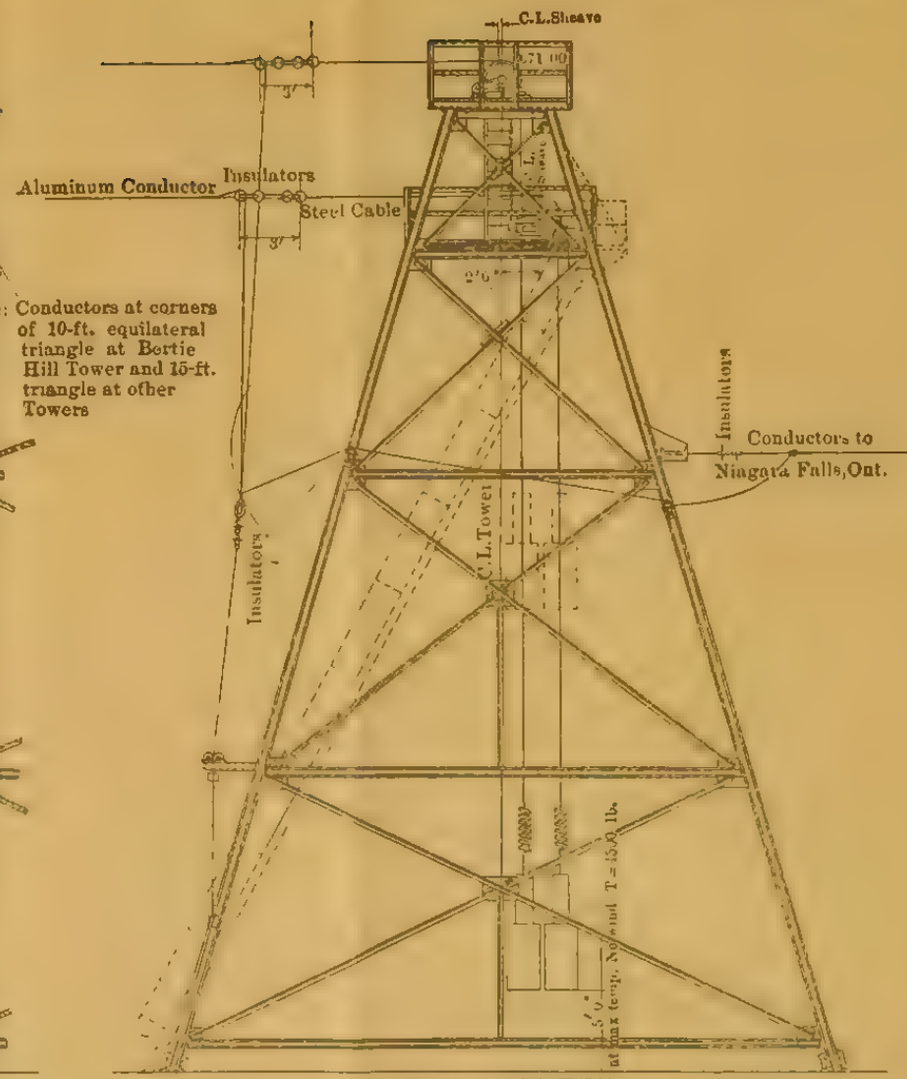
Both the pole line and the overhead crossing contain several novel features which will be described briefly. The unusual form of pole used is that shown on Plate LXXXIX. The central compression member consists of two 20-ft. lengths of 4-in. steel pipe, a cast spider with 33-in. spread being placed at the center, the pipe being stiffened with four $\frac{3}{4}$-in. sway rods. The tubular portion is stepped into a casting resting on a block of concrete. Longitudinal and transverse stresses are borne by four 1-in. guy rods attached to a casting bolted to the pole immediately below the lower cross-arm. These guy rods are anchored by concrete piers 12 in. square at the top and 18 in. square at the bottom. The soil was peculiarly adapted for this form of anchorage, as it consisted of a very hard clay which could be excavated only with great difficulty. Recent tests of an anchor, in a locality less favorable than usual, have shown that the value of the anchorage is limited only by the strength of the guy rod. The cross-arms are ship channels; the longer arm is below on the first line, and will be above on the second line. The latter arrangement is not as satisfactory, structurally, but offers certain advantages from the operating standpoint. Malleable castings are used throughout. The pins are malleable on the first line, and will be steel on the second line. The insulators are electrose and the conductors are 500 000 cir. mils, aluminum, with 37 strands. The average spacing of the poles is 300 ft., but, in the writer's opinion, this distance could be increased on tangents to 400 ft., with safety. The pole adopted is economical and durable; it is quickly erected and easily adjusted. It has no thin sections to be destroyed by rust, it requires no field riveting, and but



EAST ELEVATION

SECTION AA

Note: Conductors at corners of 10-ft. equilateral triangle at Bertie Hill Tower and 15-ft. triangle at other Towers



NORTH ELEVATION

Cable lengths are calculated on the center line distance between towers = 1669 ft., assuming the towers to be parallel

SECTION AND ELEVATION OF BERTIE HILL TOWER, AT FORT ERIE, ONT.

CANADIAN NIAGARA POWER COMPANY

little bolting. The parts could readily be galvanized, but it has not been thought necessary. Its limiting feature is the necessity for good anchorage.

Four reinforced concrete poles of unusually heavy construction are used in the Village of Chippawa, where there was not sufficient space for the standard poles; and steel structures of special construction were used at railroad crossings.

The building of transmission lines with a capacity of 50 000 h. p. across the Niagara River at Buffalo was an enterprise fraught with features both difficult and dangerous. The river at the point of crossing is a deep and rapid stream 1 650 ft. wide, through which passes all the commerce of the Great Lakes with Tonawanda—the largest lumber market in the country—as well as that to various docks in Buffalo. The cables also cross Black Rock Harbor, the Erie Canal, and the tracks of the New York Central and Hudson River Railroad. The Governments of both Canada and the United States required a minimum clearance above the surface of the river of 130 ft. The transmission lines had to pass through or over the Village of Fort Erie, Ontario, where high-voltage lines near the surface of the ground would have been objectionable. A submarine crossing would have been not only expensive in first cost and in renewals, but would have presented many difficulties of construction and maintenance, the bottom of the river being of rock and the terminal-house being about 70 ft. above the river-bed.

An aerial crossing having been decided upon, and numerous difficulties respecting rights of way having been successfully overcome, it was decided to carry the conductors upon three towers: One on Bertie Hill, west of the settled portion of Fort Erie, one at the river's edge, and a third adjacent to the terminal-house in Buffalo. The towers thus located have heights of 61 ft., 202 ft. 9 in., and 150 ft. 9 in., respectively. The tops of the higher towers are practically at the same level, while the top of the Bertie Hill tower is 107 ft. below that of the high tower. The span over Fort Erie was fixed at 1 667.57 ft. and that over the river at 2 192.64 ft.

Greater spans have been used elsewhere on transmission lines, notably at the Straits of Carquinez, yet it is believed that the combination of spans of this magnitude, with the large amount of current to be carried, had never before been attempted. Steel, copper, and

aluminum, the only materials then commercially available, were carefully considered as conductors. Steel would have been most easily erected, but its conductivity was too low; copper was rejected because, owing to its great weight, the height and cost of the towers would have been greatly increased, and the stability of the foundations of the Bertie Hill tower might have been imperiled. Aluminum was selected because of the more satisfactory relation between its weight, tensile strength, and conductivity. To obtain the greatest strength for a cross-section of 500 000 cir. mils, a 61-strand cable, made with concentric lay, was used, on the recommendation of the manufacturer. The form of lay adopted led to many difficulties which will be referred to later.

It was evident that, with conductors fixed at the ends of the span, if the minimum clearance above the river was to be preserved in summer, the towers adjacent to the river must be about 40 ft. higher than would be necessary if, by some other arrangement, the lowest point of the catenary could be maintained always at the same level; therefore, the following device was used: The conductors were fixed rigidly at the high tower on the Canadian shore, and at the American and Bertie Hill towers they pass over sheaves and thence downward to counterweights which, with the springs above them, weigh 4 400 lb., this weight being calculated to keep the lowest point of the conductors at the proper clearance line. As the temperature decreases or the wind velocity increases, the weights rise, but a constant stress in the cables is assured at all times, thus enabling them to be operated under maximum allowable stress. Plate XC shows the arrangement at the Bertie Hill tower, that at the American tower being similar. It will be seen that two pairs of spool insulators are interposed between each aluminum conductor and the steel cable passing over the sheave and connecting with the springs. The latter were added to absorb shocks, but are of doubtful necessity. The live aluminum drop-cables are attached to the main conductors by a wrapped splice, pass over a spool insulator and thence over insulators to connect with the transmission line conductors, counterweights keeping the drop-cables taut. The connection between the drop-cables and the long-span conductors was originally made with tee-clamps, but it was found that, owing to vibration, the conductors deteriorated at these tee-clamps and the wrapped splice was substituted. Experience shows that tee-clamps are to be avoided at all such connections.

FIG. 1.—ROCK AT SOUTH SIDE OF TUNNEL PORTAL.

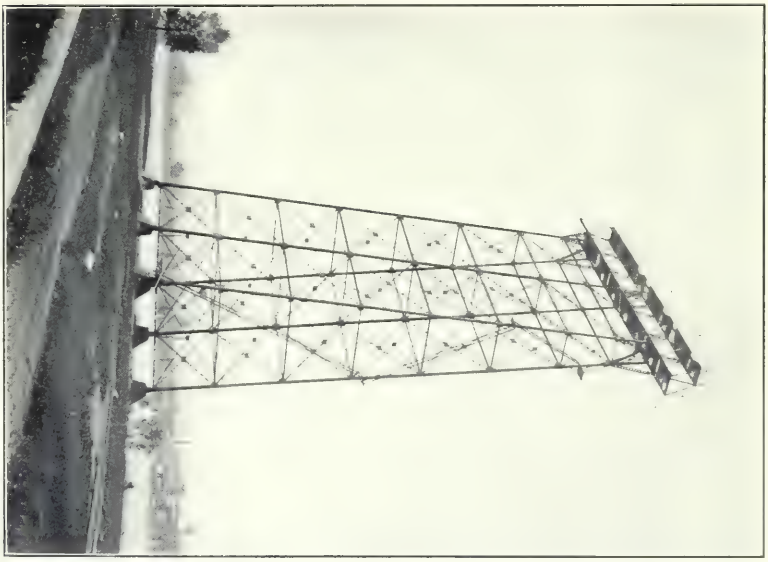


FIG. 2.—HIGH TOWER ON TRANSMISSION LINES.

Since Plate XC was prepared, bus-bars have been installed on the towers, and switches provided, so that a given pole-line circuit may be connected at will with any long-span circuit. Devices have also been added to prevent the swinging of the weights in the wind.

In the original construction the conductors passed continuously from the Bertie Hill tower to the American tower, and were rigidly secured to aluminum saddles on the high tower. However, an unexpected difficulty arose from the vibrations which occurred in the conductors when the air was still or nearly so. These vibrations were similar to those sometimes experienced in telephone or telegraph wires, but were of greater amplitude and violence. Experiments were made to dampen the vibrations, but were unsuccessful, a factor in the failure being the variation in the amplitude. The repeated stresses caused by the vibrations resulted in the breakage of strands of the conductors where they were secured to the saddles. The difficulty was finally overcome by substituting galvanized chain for about 13 ft. of conductor on each side of the saddle. The aluminum conductor passes around an aluminum sheave and returns on itself with a wrapped splice, while the chain connects with a shackle fastened to the axle of the sheave, the current being carried around the chain by an aluminum jumper cable secured to the aluminum sheaves. No trouble has been experienced since these devices have been installed.

The installation of the conductors and the various devices connected therewith, although simple in theory, proved difficult in practice, and gave rise to many complications which only experience and patience could overcome. The aluminum conductors proved very troublesome. The aluminum was so soft that it had to be guarded from every abrasion; it could not be passed under stress over snatch-blocks; it unstranded readily; and yet it had to go out in perfect condition, as it could not be inspected after erection, and had to operate at a high tensile stress. The rope lay would have been far easier to handle, but the manufacturers thought that a given strand might cut into the adjoining strands under stress. The method of erection finally used was as follows: Telephone service was established between the three towers, flag signals having proved unsatisfactory. Two unusually large hoisting engines were utilized, one at the American tower and one at Bertie Hill tower, from which all the conductors were drawn. A manila rope was first taken across by a tug, the swift current

and the frequent passage of vessels rendering this an operation of great difficulty. The rope was used to draw a steel cable consisting of two pieces and having a length somewhat greater than twice the span. The front drum of the Bertie Hill engine was lagged to form a "nigger-head," about 3 ft. in diameter, and was covered with grease.

The reel of aluminum cable having been placed on a special stand provided with a brake, the end of the cable was passed eight times around the drum and thence over a large wheel to its proper position at the top of the tower. A long wooden clamp was then bolted around it near the end, and this was secured to the end of the steel cable connected with the hoisting engine at the American tower. The steel cable leading from the back drum of the Bertie Hill engine was then passed through the opening for the next cable and secured to the other steel cable near the aluminum conductor. The engine on the American side having started up, the aluminum was allowed to pass off the "nigger-head" while the steel cable unwound from the back drum. Thus the single steel cable on the American engine drew over both the steel and aluminum cables.

It was necessary that the two drums on the Bertie Hill engine and the American engine four-fifths of a mile away should work in unison, for if the cables sagged too much, the masts of vessels would be struck, while, if the aluminum conductor was pulled too taut, it would be broken. When the head of the cables reached the high tower, the rear steel cable was disconnected under stress 200 ft. above the ground and carried through the lattice work of the tower in the correct position for the next cable and again secured to the leading cable. The return trip was a simple one, but, on the whole, the work involved much anxiety, owing to the high stresses under which the cables were drawn and the serious consequences which might have attended an accident. In future work of a similar nature, the lot of the engineer will doubtless be a happier one, owing to the use of copper-plated steel-wire conductors or other of the recently produced transmission cables.

The conductors are operating successfully, and are daily carrying to Buffalo large amounts of power for the operation of industries and public service utilities. During a recent sleet storm, which practically doubled the weight of the cables, the apparatus operated exactly as de-

signed, and the conductors were not injured, although, if they had not been counterbalanced, injury to them would have been certain.

Fig. 2, Plate LXXXVI, is a photograph of a portion of the transmission line between Niagara Falls, Ontario, and Fort Erie, and Fig. 2, Plate XCI, is a view of the high tower, erected upon the Canadian shore of the Niagara River, carrying the conductors to the terminal station in Buffalo, N. Y.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SUBSTRUCTURE OF PISCATAQUIS BRIDGE, AND ANALYSIS OF CONCRETE WORK.

Discussion.*

By Messrs. George B. Francis, James H. Brace, A. L. Bowman, James Owen, and James C. Boyd.

Mr. Francis.

George B. Francis, M. Am. Soc. C. E.—About six years ago, some concrete engine and generator foundations were built under the speaker's direction, one of which it was necessary to remove after it had been standing two or three months, and in the removal there was opportunity to observe the effect of the stoppage of the placing of the concrete over night—the original construction occupying several days.

This particular foundation was approximately 30 ft. long, 20 ft. wide and 15 ft. high, and the cause of its removal was a change of the type of engine and generator which it was to support, the new ones requiring a totally different arrangement of anchor bolts.

The concrete was supposed to be a monolithic structure, and it had become so thoroughly set that it required the use of dynamite to break it up.

In separating, it came apart on horizontal lines corresponding to each day's work, showing distinctly that there was only moderate cohesion of one day's work with that of another, although there was no evidence of these seams on the exterior surface. The vertical breaks, as might be expected, were ragged and through the homogeneous mass.

In recent work it has been the custom to place the concrete for such structures without intermission until completed.

* This discussion (of the paper by G. A. Hersey, Jun. Am. Soc. C. E., printed in *Proceedings* for March, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hersey speaks of using some sort of a metal shoe on all piles for foundation work. The speaker does not agree with him in this, as there are many cases, where piles are required, where it is quite plain that they will have to penetrate known strata, and that a metal shoe or protection is unnecessary and a great waste of money. Mr. Francis.

In many cases where piles cannot be driven without shoes, it is pretty good evidence that they are not needed. When they can be driven only 4 or 5 ft., it is better to deepen the excavation and not use them at all.

James H. Brace, M. Am. Soc. C. E.—In the speaker's practice, the question of joints has come up in the construction of arches. In one case (a very flat, thick, and heavily reinforced concrete arch), as the contractor did not wish to work at night, the arch was divided into voussoirs by radial bulkheads. The size of each voussoir was about equivalent to one-half the daily capacity of the concrete plant. Work was started at the skewbacks, and one section on each side of the arch was filled during the day. This secured a balanced loading on the centers, and radial joints. The results were satisfactory. Wherever possible, however, the speaker believes it is best to work continuously until the arch is completed. When this is done, it is well to build up the concrete as nearly as possible along radial lines, in order to avoid bad joints in case of some unavoidable delay. This is particularly important in lining tunnels, as the work approaches the top of the arch. It always requires much more time to place the concrete in the key, and the concrete on either side is apt to take considerable set before the work is completed. Mr. Brace.

It is thought that by "whitewashing" the author means a layer of neat cement grout brushed over the finished work. The speaker has seen this used, and thinks that it generally improves the appearance of the work. It will sometimes peel off, but is more apt to adhere to the wall than a plaster coat.

A. L. Bowman, M. Am. Soc. C. E.—In work of this kind, to secure the best results in quality and appearance, a wet concrete should be used, and to secure the advantages of a monolithic structure, within certain lines where possible, the concreting should be carried on continuously, day and night, until finished. Mr. Bowman.

A few years ago the speaker had occasion to build some reinforced concrete railroad arches. The abutments and piers were carried up to the springing lines and there stopped. When the arch rings were started the work was carried on continuously, night and day, until finished.

In one of the spans—a double-track, 54-ft. skew arch—the concreting was started simultaneously from each skewback, and was built in 60 hours of continuous work. The concrete was mixed very wet, and, owing to the number and location of the reinforcing rods, very

Mr. Bowman. little tamping could be done. The forms were made of dressed lumber, and the inside faces were coated with "Petrolatum" as soon as erected, a second coat being applied just before the concreting was started.

To secure a smooth surface, the concrete was worked thoroughly against the forms with forks, so as to force the stone back into the body of the ring and flush the cement mortar to the surface. The forms were set $\frac{1}{2}$ in. high at the crown, and, after the arch ring was completed, were left in place for about two weeks, after which they were lowered by small amounts every few days until they were clear of the arch.

When the centers were entirely removed, the inner face of the arch presented a very smoothly-finished, uniformly-colored surface, which only had to be touched up and rubbed down in a few spots.

There were no hair cracks visible, and very satisfactory results were secured without using any cement wash or whitewash coat.

Mr. Owen. JAMES OWEN, M. AM. SOC. C. E.—In the construction of piers and concrete work there is one feature which has not received much attention, although it has been a matter of some consideration, and that is what is known as the over-night bond. It is a generally accepted fact that it is very difficult to obtain a perfectly homogeneous connection with work finished over-night and built on the next day, on account of expansion. In a large proportion of concrete construction, the joint, where one section is put upon the other, is very noticeable when the work is completed, and many methods of overcoming this difficulty have been tried.

The speaker tried the experiment of making the last run of the machine very wet, and found this somewhat efficacious. This run was so wet that the next morning the water had all collected on the top of the pier, with very slight setting on the surface of the concrete. The water was swept off and the new joint made, and thus the difficulty was overcome, in a measure; but, if there are such joints in pier construction between high and low water, such trouble is likely to occur.

A great many engineers do not allow any intermission, the work being carried on continuously night and day. This method eliminates entirely the bond question, but it is somewhat more costly.

The speaker's attention was first called to the bond question in the case of a Paterson bridge which was carried away by a freshet. On examination, the break was found to have occurred on a line of work where two sections had not been thoroughly bonded in construction.

Mr. Boyd. JAMES C. BOYD, M. AM. SOC. C. E. (by letter).—Some years ago the writer was in charge of bridge construction for the Bangor and Aroostook Railroad, and therefore he has read with much interest Mr. Hersey's paper on the Piscataquis Bridge. At that time a bridge, very similar in character and construction, was built at what is known

as the Second Crossing of the Fish River. The close similarity in some of the results obtained on the Fish River and this Piscataquis work is rather striking. On the Fish River Bridge, the concrete was mixed by hand, the aggregate being bank gravel taken from a near-by cut. This gravel was not screened, but was tested in a rather crude manner, under the writer's direction, by periodically taking sample buckets of the gravel and ascertaining the voids by noting the quantity of water required to fill them, a correction being made in the aggregate by the addition of either sand or gravel as the case required. In the concrete it was desired to obtain the equivalent of a 1 : 3 : 5 mixture, and the records of this work show that two bridge abutments, containing 1 668 cu. yd., required 2 034 bbl. of cement, giving an average of 1.21 bbl. of cement per cubic yard of concrete. This work was performed by day's labor by the railroad company, and the cost, exclusive of overhead charges, but including foundation caissons and pumping, forms and tools, was $7.76 per cu. yd. of concrete, cement costing $3.00 per bbl. delivered at the site. Mr. Boyd.

In a similar bridge, known as the First Crossing of the Fish River, built at the same time and in the same manner, 3 216 cu. yd. of concrete required 4 060 bbl. of cement, giving an average of 1.26 bbl. per cu. yd. of concrete.

The author states that, for absolute safety, it is necessary to furnish all piles with point protection. The writer feels that this is not strictly true, although, in material found in the river bottoms in the northern sections of Maine, it is no doubt the safest course to pursue. However, in many places where soft material is found and the piles depend on skin friction for their supporting power, as well as in many places where the character of the foundation is well known, it would hardly apply.

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THE MINING OF METALS.

An Informal Discussion at the Annual Convention, June 25th, 1908.*

By Edwin N. Hawkins, Esq.

Mr. Hawkins.

Edwin N. Hawkins, Esq.†—Colorado mining methods and procedure, and the efficiency of its miners, are everywhere synonyms for excellence and intelligence in the field of mining. This is the result of long years of schooling in battling with the problems the industry has presented in this State. No other commonwealth has encountered an equal number of problems for the profitable treatment of refractory ores. Starting with the first lode mining in Gilpin County, the pioneers speedily came to the sulphides, which baffled their attempts to win the gold and silver by the only method then known, amalgamation with mercury. It required several years for even small progress toward rendering the metals partially free by roasting the sulphides containing them. The building of the smelting works at Black Hawk, in Gilpin County, thus marked an era of advancement. Following closely, the Leadville District was opened, together with several minor camps, and at once it became necessary to apply smelting methods to recover the values. Placer mining had preceded both in this and the Gilpin section, having had its birth with the first discovery of gold in the State, on the banks of Cherry Creek, within the City of Denver.

Even with the application of shaft-furnace smelting in Leadville, however, the problems of the necessary recovery, to render mining

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

If additional communications on this subject are received, they will appear in a subsequent number of *Proceedings*.

† Member, American Institute of Mining Engineers.

profitable, were only beginning to be met. Trained metallurgists, schooled in Germany, were the initial brains and energy, and the leaders in the infant industry. They were soon joined by young men from technical schools, and the improvements made in the recovery of values were rapid. Thus progress in metallurgy has been intimately and constantly connected with the growth of the mining industry from its very beginning in Colorado. Mr. Hawkins.

The ore products, first valued for their gold and silver contents only and paying the miner less than what would now be an enormous treatment charge, were of necessity high in grade. Gradually the contained lead and copper were paid for, and added to the gross value of the ton, and finally the miner began to share also in the value of the more desirable fluxing constituents of his ore product. That which in 1879 was a small weight in tons and high in grade was added to in the growth of ten years by many thousands of tons monthly, which brought the producer upwards of $10 more per ton in first or gross valuation, by reason of these new features and an accompanying reduction in the treatment charge as well. The progressive smelting establishments grew in tonnage, importance, and profit; and the development of some of the newer districts furnished them with the necessary variety of ores to stimulate still further the tonnage production of the mines, together with their own growth and prosperity.

Thus the pioneer solution of mining and metallurgical problems has made the rank and file in the State's mining industry the leaders in methods, production, and both mechanical and metallurgical perfection.

The coal product and the transportation facilities of the State have played important parts in its mining history. Its districts have been quickly rendered easy to reach by rail, and the most speedy development has been made possible. At no time has the industry or its metallurgical sister been dependent on resources without the State boundaries. In due course of electrical advance, Colorado was the first State to apply this force to mining operations. The application of electrical power to mining dates back to installations in Aspen, first for mine hoists and later in this camp, as well as generally in most of the other mining districts in the State, to pumps, blowers, air compressors, mine and mill haulage, shops and motors, with the power for these latter applied also directly at the point of its use upon a particular machine. Water-powers in Colorado were developed for electrical conversion and subsequent distribution to the mines. Where such powers are not available, central power-plants are using coal.

In the high, mountainous sections of Southwestern Colorado, the successful operation of a number of large mines is the direct result of the utilization of a more or less distant water-power and its electri-

Mr. Hawkins.

cal conversion and transmission to the point of desired application in both mine and mill. In such cases solid fuel could not have been transported and used except at such an enormous expense that loss would have been the net result, instead of profit. With the growth of particular mining districts, the application of the current has made possible the use of lengthy individual or community development, and transportation tunnels and drifts to cross-cut or open veins at greater depths, with a consequent delivery of the ore product nearer to treatment mills or transportation facilities. It is true that the first few years of the application of electricity to mining showed small progress; but it was entirely due to two factors: The uncertainty naturally felt in the use of a new power, and the time requisite for manufacturers to learn the needs of the industry and become acquainted with the conditions to be met in underground work. Electric drills present the only problem not yet solved by the current with entire satisfaction, but progress in these machines in the past few years has been such that the difficulties, which are altogether of a mechanical nature, will undoubtedly be overcome.

Probably no section of the State, with the exception of the mining camps of Southwestern Colorado, presented as favorable a field as the Cripple Creek District for the use of the electric current in mining operations. There, even though, close to the south, there were large producing fields, the considerable expense of coal furnished ideal conditions for the manufacture and transmission of the current from centrally-located stations at or close to the coal mines. The great monthly ore tonnage of the district from both the larger mines and the many individual lessees, working smaller blocks of mining ground, has made a ready market for electrical power, and its use, particularly for operators of the latter kind, has been indispensable. In no other way could many of the lessees of the district have opened the same amount of profitable ore in the limited time allotted by their lease. In this manner, not only have the normal growth and production of the camp been fostered, but the frequent consequent reduction in the mining cost per ton of ore has actually added a tonnage of ore which could have been made available in no other way. The system of rental of motors has also reduced the initial outlay of the lessee operator, and hence the benefit derived has been shared by the manufacturer of the current as well as the lessee. The Colorado gold production in 1907 was $20 888 833. In 1906 it was $22 934 400, a decrease of $2 045 567 in the last calendar year. Much or all of this decrease was caused by the destruction by fire of the largest ore treatment mill in the State, which was operating entirely on Cripple Creek ores. This calamity necessitated the suspension of contracts for a period of five months. The reconstruction of the mill was completed in January, 1908.

Mr. Hawkins.

The mining of silver is connected more intimately than gold with ores carrying also the base metal values of lead, zinc or copper, or any association of them. Some of these, indeed a large proportion, either require concentration by putting any possible given number of tons into one, or they become more profitable thereby. Most of the treatment of this character is applied by the mine at its own plant, and the product is shipped to smelting centers. In other cases, custom plants either purchase the mined products outright, or charge the shipper at a tonnage rate and return him the concentrated product for his own disposition. The crude and concentrated products, carrying silver and lead values, are reduced to metal at the smelter center. In 1907 the silver product of Colorado was 11 648 136 oz., Troy, having a value of $7 687 769. In 1906 it was 12 447 400 oz., Troy, having a value of $8 185 276. The decline in the price of the metal was sharp during the last five months of the year, in common with the prices of lead, zinc, and copper. The total fluctuation—between 68 and 52 cents per oz.—was marked by a drop of 10.68 cents per oz. from August to November, 1907. A natural curtailment of production resulted, and was increased by a restricted market for lead ores during a portion of the year.

The lead-ore production of Colorado is associated with silver occurrences, and is treated altogether by the smelting establishments. During 1907 the production was 50 000 short tons, having a value of $5 111 000, while the tonnage of 1906 was 51 000 tons, having a value of $5 756 658. During 1907 the price of the metal fell from 6 to 3.65 cents per lb. The drop in prices, resulting finally in the abandonment of any effort to maintain a fixed rate and an open market throughout the country, caused the smelting establishments to discourage the mining and shipment of lead ores for the last six months of 1907, except where contract obligations prevented.

When zinc accompanies lead in silver ores, usually to an extent in excess of 8 or 10%, it frequently renders the ores unprofitable as to their other metal value, and in such cases the electric current, again, has been applied advantageously, and a separation made of the complex ore into three products, namely, zinc sulphide, lead sulphide (both as concentrated products), and the comparatively valueless quartz or rock waste. In the past few years a very large industry has been adding to the State's production—chiefly from Leadville—by this method of separation and concentration. In 1907 the production of zinc from ores was nearly 130 000 tons, having a value of $5 000 000, of which 5 332 tons was spelter, and practically all of it was thus obtained, giving two valuable products from a formerly valueless one.

The copper production of the State is of comparatively little importance, being only a little more than 1% of the whole country's total. The building of new railroads will open virgin copper and gold

Mr. Hawkins.

fields during 1908 and 1909. The production for 1907 and for 1906 has been about 5 000 tons of copper yearly.

The production of metalliferous ores has received, and will always receive, a very large proportion of the benefits and savings arising from the advancements of the metallurgical art. Prosperity, therefore, is directly increased, and, in return, there is mined a greater tonnage of ore. Even though the grade of the average be lower, the net profit is increased; hence the mine, the treatment plant, and the labor employed all share. In a comparatively short time, smelting charges, on much of the ore product of the State, have been reduced, and in many cases rates have been established for the especial purpose of stimulating the production of the fluxing ores, carrying lead and iron in important quantities. These advantages and encouragements have also been extended to shippers of silicious or quartzy ores of the Cripple Creek District, and the tonnage produced has responded accordingly. The mills, however, have been the most important factor in this respect. The chlorination treatment of Cripple Creek ores, first extensively and successfully applied, has been still further improved, chiefly in the cost per ton, by fine grinding and cyanidation of the roasted ores. One large plant is reducing in excess of 20 000 tons monthly from the mines of Cripple Creek. A portion of this quantity is ore having a gross value of less than from $8 to $12 per ton, and the mere fact that such low-grade ores return a profit to both the producer and the reducer, where it was formerly impossible to mine them, speaks eloquently of metallurgical progress in this particular field. Ores of still lower value, even such as are being taken in bulk or sorted from what were formerly waste mine dumps, are now being treated in a large way at the new Independence Mill and others. The absence of necessity for roasting these very low-grade ore-dump tonnages reduces the cost so much that it is likely that this practice will be extended to other mines and dumps before the year is over. Metallurgy, therefore, has been a most important adjunct in the progress of a section which still continues to be the largest gold-producing mining camp on the Continent.

The use of the slime treatment and the filter press, both with tube-mill grinding and the cyanidation of gold and silver ores, is yet to come in Colorado, and remains to be developed upon some appropriate ores. The treatment is speedy, the cycle is continuous, and hence the tonnage handled will be so large that the low cost per ton will give an attractive profit. In numerous instances in Clear Creek and Gilpin Counties concentration mills have made important improvements in their mechanical handling and their percentage of saving. In these counties there has been the same value in ore production for each of the past two years, with the advantage, however, that in the past twelve months an unusual amount of development work has been ac-

Mr. Hawkins.

complished by the application of the cheaper electrical power transmitted from a central station. As a result of the year's improvements in metallurgical practice, the State will have, from these long-established and substantial mine districts, a largely increased production of both ores and values in gold and silver, as well as in concentrates containing lead and zinc.

The progress of Colorado in a few years in zinc metallurgy has been noteworthy. In spite of the first high cost of plants for magnetic separation, much has been and is being done. Besides the Wetherell separator, which has been the pioneer in this field, the Blake patents, together with important improvements acquired by the Blake Company, have successfully applied the electro-static current to the separation of the sulphides of lead and zinc, and have entered a field not open to the Wetherell separator, but between them they cover all classes of zinc sulphides. Recently, Blake improvements have shown ability to make a closer split or saving than before, and the speedy application of these machines to the general sulphide product from Leadville, Clear Creek, and other districts, may be expected. A. R. Wilfley has devised and made successful the application of a rapid magnetic or magnetizing roasting of sulphide ores, followed by magnetic separation of the product into concentrates and waste. There are in the State already six such plants, built and building, having a total daily capacity of 700 tons of crude ore. Thus the value of the zinc ore and metal product of the State is rapidly approaching that of its lead.

Still another method, as yet more or less in the experimental stage, however, promises to become a useful adjunct to the mining and successful handling of low-grade lead and zinc sulphides. It is the Elmore process, and is based on the action of dilute acids on crushed low-grade sulphide ores, whereby hydrogen is produced. The gas acts as a float for the sulphide particles, and they are withdrawn by a vacuum suction, dried, and subsequently treated. Enough has been done to indicate eventual commercial success.

It is to be noted with much satisfaction that the leasing system is again being extended in the Cripple Creek District. In the past, this method of development has been most productive of tonnage and value. It is the lessee who makes the most of the new discoveries. All the mining districts of the State need the system. The Northern District, comprising Clear Creek and Gilpin Counties, Leadville, and the Southwestern sections are applying it, to a considerable extent, from all of which material results may certainly be expected.

Gold dredging in Colorado is practically confined to Summit County, on the Blue and Swan Rivers, where two companies are actively at work with several dredges built during the winter of 1907–08.

Mr. Hawkins. The great variety of Colorado's mineral production covers nearly all fields of engineering and material needs. For some years past, ores of tungsten have been produced, chiefly in Boulder County. In 1906 the State's product was valued at $225 000, in 1907 it reached double that figure, in spite of the complete cessation of purchases of tungsten concentrates during the last two months of the year. Activity in this industry is again established. The largest portion of the country's product of tungsten is mined in Colorado, and the successful development of the new tungsten lamp will create a still larger demand than has heretofore come from the steel industry alone.

Several years ago attempts were made to produce oxides of vanadium and uranium from the sandstone deposits of San Miguel and Montrose Counties, in Colorado. Investigations have shown the presence of these oxides in commercial quantities, and plans are now under consideration for the concentration and shipment of such products to Pittsburg and to Germany. Steel makers are always in the market for these oxides, and the unit price offered is sufficiently high to stimulate production.

The published dividends from metalliferous mining in Colorado were $4 225 000 for the year 1907. The total metalliferous production referred to herein was $40 000 000 in the same year. Other mineral products brought the State's total production up to $94 000 000. Progress thus far in 1908 is at a rate in excess of these totals.

The present railroad construction of the Denver, Northwestern and Pacific is about to enter a section of Northern Colorado from which the State will soon realize increases in gold and copper production as well as an immense addition to its coal output. Central power-plants are under extensive construction in the northern and southern parts of the State, and their completion will mean a great deal to the miner, who will then be able to secure the application of power where the cost of coal has heretofore made operating prohibitive.

No labor troubles disturbed the State, as in former years, and had it not been for the comparatively small effect of the recent financial disturbance on the State's ore and metal productions, felt chiefly during the last six months of 1907, the year would have shown the usual normal gain of 5% or more. The mining wage scale is high, generally, and the miners are in the main satisfied. Industrial peace reigns, and it will not be soon disturbed. Regularity of work has characterized mine operations for several years past, with the exceptions already referred to. The business upheaval during the autumn of 1907 was not felt to any extent. No condition of unusual prosperity had reigned previously, and, while some sister States were keenly affected, Colorado was disturbed very little. It is true, that, as compared with 1906, the gold production fell off 10%, and that the dividend record was slightly reduced, but it is also true that 1907 saw

little or no reduction in the number of tons of ore produced, or in the number of hours of employment of the wage earner in mining. What might be called an investment basis in the State's mining operations was reached. Speculation has been largely eliminated, and, from the mining fields, one may confidently look for the regular results to be expected from progressive metallurgical improvement, accuracy, and energy. Mr. Hawkins.

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THE EFFECT OF TEMPERATURE CHANGES ON MASONRY.

Discussion.*

BY MESSRS. GEORGE G. HONNESS, THADDEUS MERRIMAN, AND WILLIAM LOWE BROWN.

Mr. Honness.

GEORGE G. HONNESS, ASSOC. M. AM. SOC. C. E.—The speaker is glad that the subject of the effect of temperature changes on masonry has been brought before the Society, and wishes to add his observations of such effects on large masonry structures which have been under his supervision, in the hope that such information may lead to methods of caring for the resulting temperature cracks.

In addition to the more general use of Portland cement and the resulting character of masonry, it seems likely that the greater rapidity of construction attained in recent years is another factor which has led to the more frequent and noticeable development of temperature cracks, because of the probably greater range of temperature caused by the retention of the heat generated by the setting mass of masonry, which heat must, of necessity, be retained for longer periods.

As an example of the progress of laying masonry a few years ago, compared with the present practice, the following is cited:

"At the Sodom Dam, near Brewster, N. Y., in the Croton water-shed, the maximum monthly progress was 3 000 cu. yd., this being rubble masonry, laid partly in American and partly in Portland cement. At the Titicus Dam, at Purdys Station, N. Y., also in the Croton water-shed, the maximum monthly progress was 5 700 cu. yd., the character of the masonry being the same as at the Sodom Dam.

* This discussion (of the paper by Charles S. Gowen, M. Am. Soc. C. E., printed in *Proceedings* for April, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

FIG. 1.—DOWN-STREAM FACE OF CROSS RIVER DAM.

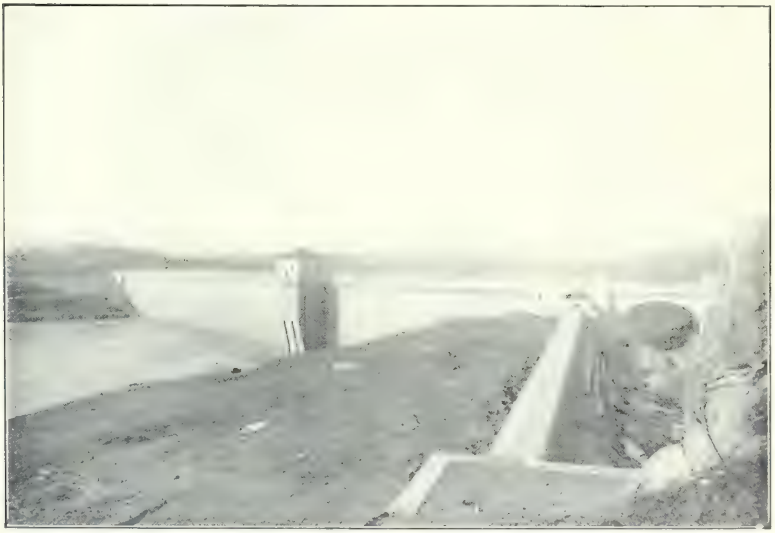


FIG. 2.—UP-STREAM FACE OF CROSS RIVER DAM.

REINFORCING RODS IN THE CROSS RIVER DAM.

Mr. Honness.

At the Boonton Dam, Boonton, N. J., the maximum monthly progress was 21 000 cu. yd., and at the Cross River Dam, Katonah, N. Y., the maximum monthly progress was 18 400 cu. yd."

The two structures of hydraulic masonry which the speaker has had an opportunity to observe closely are the Boonton Dam and the Cross River Dam.

The Boonton Dam.—The Boonton Dam, at Boonton, N. J., is built of cyclopean masonry, the down-stream face being ashlar, above the lines of refilling, and the up-stream face, rubble masonry. In all, about 255 000 cu. yd. of masonry were required for this structure, Portland cement being used exclusively.

In the season of 1902, from May 20th, 84 000 cu. yd. were placed; in 1903, 134 000 cu. yd., and in 1904, 37 000 cu. yd.

No cracks in the dam were discovered in the winter of 1902-03, which is probably accounted for by the fact that the masonry laid in the season of 1902 was all placed in the lower portions of the dam section where the width was great enough to withstand the stresses due to the ranges of temperature, and, to some extent, for the additional reason that a layer of masonry was laid over the entire portion of the dam then under construction in the colder months.

In the second and third winter seasons, however, numerous vertical cracks developed. There was no regularity in the distance between the cracks, and in no place were there any evident signs of horizontal cracking. The "racks," which seemed to be unavoidable, apparently had no influence on the location or the number of cracks. The largest cracks were near the ends of the dam, and at least six of them leaked a considerable quantity of water, which leakage materially diminished in the summer months, but did not cease entirely.

No effort of any kind was made to treat these cracks or to diminish the leakage.

By measurement and estimation, the total width of all the cracks on top of the dam was $3\frac{1}{2}$ in., and the largest ones could be traced down the face of the dam for a distance of 60 ft. from the top.

Thermophones were embedded at different points in the dam section, and some interesting data regarding the interior temperatures of a large mass of masonry were obtained. Thaddeus Merriman, Assoc. M. Am. Soc. C. E., who was Principal Assistant Engineer on this work, has made an analysis of the data obtained from these thermophone records and of the temperature cracks in this dam.

Cross River Dam.—The Cross River Dam is built of cyclopean masonry and faced, both up stream and down stream, with concrete blocks. Fig. 1, Plate XCII, is a view of the down-stream face from the south end, and Fig. 2, Plate XCII, of the up-stream face, showing the waste-weir and gate-house.

Mr. Honness.

In all, 158 000 cu. yd. of masonry were required, Portland cement being used exclusively.

The upper 30 ft. of the dam was reinforced with 1¼-in. square twisted rods, or rods of equivalent area. They were placed in six horizontal layers, laid longitudinally and hooked together at the ends by right-angled bends. Plate XCIII shows the rods of the second layer in place, and Fig. 2, Plate XCV, their location. The number and position of these rods were fixed by the Chief Engineer, Walter H. Sears, M. Am. Soc. C. E., and W. H. Burr, M. Am. Soc. C. E., of the Aqueduct Commission. It was not expected that the use of these rods would entirely eliminate the temperature cracks, but it was hoped that it would prevent their concentration in a few large cracks.

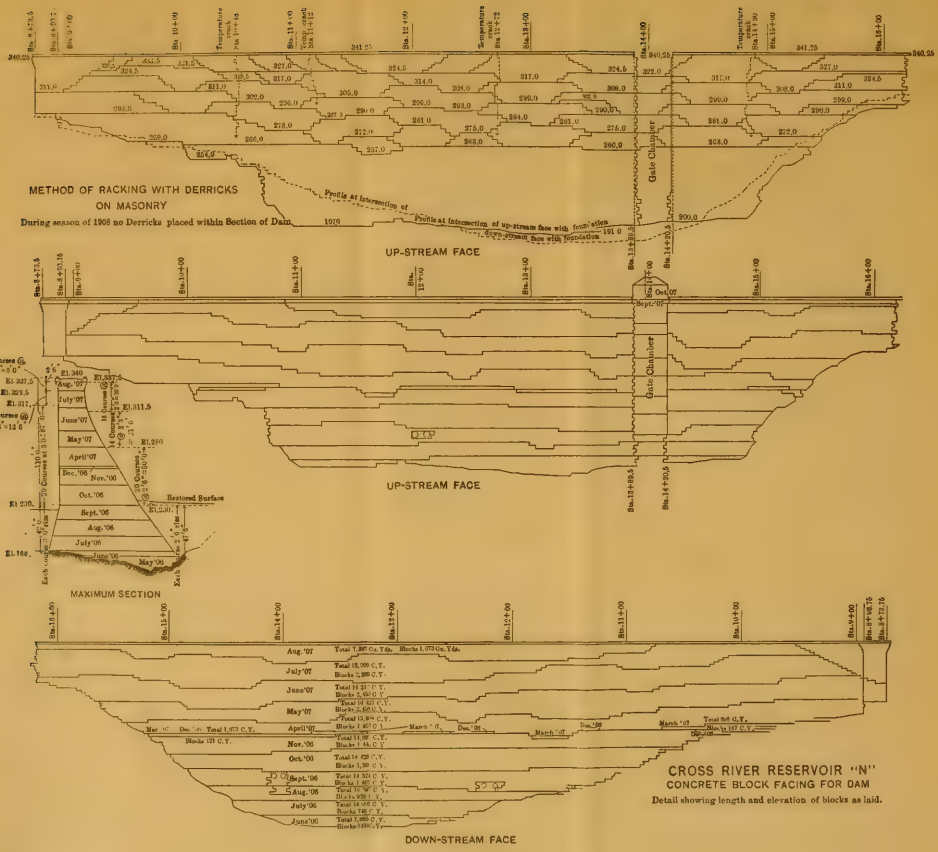
In the first season, about 87 800 cu. yd. of masonry were placed, the maximum month's work being 18 400 cu. yd. During the first season, the dam was built in approximately horizontal layers by derricks placed outside the section. Work was continued until about the middle of December, when it was suspended because of cold weather. By this time the dam had been built for a height of about 92.5 ft. above the lowest point in the foundation, the maximum bottom width being 116.3 ft. and the top width, 51 ft. No temperature cracks were discovered during the winter of 1906-07.

During the second season, 1907, the dam was built with derricks placed directly on the wall. These were arranged back to back, and built a pinnacle with a maximum height of 9 ft., the "rack" being on a 1 to 1 slope, this being the limit of maximum height and slope allowed. Upon the completion of these pinnacles, the derricks were shifted to a similar position on the top of the pinnacle, the intervening space was filled, and another pinnacle was built thereon. This operation was continued until the dam was completed. Plate XCIV shows the order in which the masonry was placed, and the quantity for each month, also the manner of placing the masonry and the location of the cracks.

Work was resumed late in March, and the main dam was completed by the end of August, 1907; consequently, all masonry used in its construction during the season of 1907 was laid in the warmer weather.

The waste-weir, the section of which is about 9 ft. high and 9 ft. wide on top, was built mostly in September and October, in sections 35 ft. long, a slip-joint being provided at the end of each section.

The first temperature cracks in the main dam were noticed early in October, 1907, and were four in number, located at Stations 10 + 46, 11 + 08, 12 + 73, and 14 + 89, as shown on Plate XCIV. By March 1st, 1908, three of these cracks could be traced down the face of the dam for a distance of 70 ft., and the fourth for a distance of about 42 ft. from the top. There was no indication of any horizontal cracking, except at points where the cracks for short distances followed



CROSS RIVER RESERVOIR "N"

CONCRETE BLOCK FACING FOR DAM

Detail showing length and elevation of blocks as laid.

the horizontal joints of the blocks. The cracks were widest at the top, gradually diminishing in width toward the bottom. The maximum width of all the cracks was $\frac{3}{4}$ in. Mr. Honness.

No leakage or seepage through the dam or through the cracks was noticed until January 16th, 1908. On this date the water stood at a point 53 ft. below the top of the dam, and the leakage was noticed at a distance of 70 ft. from the top of the dam. Fig. 1, Plate XCV, shows the crack at Station 14 + 89, which extends through a header block. A slight amount of leakage continued until February 7th, when it suddenly increased, indicating that there was a free passage for water through the dam section at the cracks. On February 10th, a measurement showed a leakage of 6.6 gal. per min., and by February 17th, this had increased to 23.9 gal. per min., which is the maximum amount of leakage measured at any time. As soon as the marked increase in leakage was noticed, steps were taken to control it. This was accomplished by caulking the crack at the up-stream face with lead wool and grouting the crack. These measures were successful to the extent of reducing the leakage to 4.5 gal. per min., and this has continued to diminish since the warm weather set in.

From these observations it would appear that the dam was entirely cracked through for a distance of about 43 ft. from the top, at which point the section is 31.75 ft. wide, and partially cracked for a distance of 27 ft. further from the top of the dam. This condition, and also the position of the reinforcement, is shown by Fig. 2, Plate XCV.

In the waste-weir, all slip-joints opened, but not to as great a width as the cracks in the main dam, probably due to the fact that this masonry was laid in the colder months.

From the observations made on the structures here described, the following conclusions are drawn:

1st.—While the speaker does not wish to be understood as favoring "racks" in hydraulic masonry, believing them to be objectionable, it does not appear that they, in any way, affect the number or location of cracks.

2d.—It does not appear that there is any extensive separation of the masonry on horizontal planes. The vertical cracks seem to be caused by the expansion and contraction of the molecules of the masonry itself, due to ranges of temperature.

3d.—There is apparently no uniformity in the distances between cracks. At Boonton, there is a crack for about every 100 ft. in the length of the dam, and at Cross River, one for about every 200 ft. This may or may not be due to the manner in which the masonry is placed, the irregularity in the profile of the foundation, or to the length of the dam.

4th.—The reinforcement in the Cross River Dam—as to quantity and method of placing—was not effective in preventing the concen-

Mr. Honness.

tration of the cracks. Another arrangement and a greater quantity, carried for a greater distance from the top of the dam, might prove beneficial, and distribute the effect of temperature changes in a large number of very small cracks.

5th.—It would appear that it would be best to provide for the control of the temperature cracks by the use of vertical joints, as suggested by Mr. Gowen, not overlooking, however, the fact that it is practically impossible to prevent a slight seepage, which will disfigure the face of any dam, no matter what class of facing be used. Provision should be made to prevent this seepage from reaching the down-stream face. Experience alone will show what details should be used in making these joints, but it seems certain that they will not add materially to the cost of construction.

Mr. Merriman.

THADDEUS MERRIMAN, ASSOC. M. AM. SOC. C. E.—This subject is of interest to every engineer. It is, moreover, one which, at the present time, is but little understood. As more facts concerning it are brought out, therefore, they will lead to a better knowledge of it and, consequently, better design of masonry structures.

The author has presented one phase of the subject. The speaker will endeavor to present another, basing his conclusions on the observations of the interior temperatures in the masonry dam at Boonton, N. J.

The Boonton Dam was built during the years 1902, 1903, and 1904, across the Rockaway River, near Boonton, N. J., by the Jersey City Water Supply Company, for the purpose of furnishing a supply of water to Jersey City, N. J. Its design and construction were under the direction of Edlow Harrison, M. Am. Soc. C. E., Chief Engineer, J. Waldo Smith, M. Am. Soc. C. E., Consulting Engineer, William B. Fuller, M. Am. Soc. C. E., Resident Engineer, and George G. Honness, Assoc. M. Am. Soc. C. E., Division Engineer.

The total length of this dam is 3 100 ft., 2 150 ft. being of masonry having the cross-section shown by Fig. 2. The maximum height of the full masonry section is 114 ft., and its average height for its entire length is 90 ft. At its north end the masonry is 39 ft. in height, and at its south end the top of the dam is 67 ft. above the foundation. The axis of the dam bears 12° west of north, and the dam impounds water on its west side. The profile, Plate XCVI, shows the progress on the masonry during construction, as well as the locations and sizes of the temperature cracks which have occurred. Fig. 1 indicates the number of cubic yards of masonry built during each month.

The down-stream face of the dam was faced with ashlar, as shown by Fig. 1, Plate XCVII. The up-stream face was formed of rubble, as shown by the photograph, Fig. 2, Plate XCVII, which also gives a view of the intake gate-house. The general character of the interior

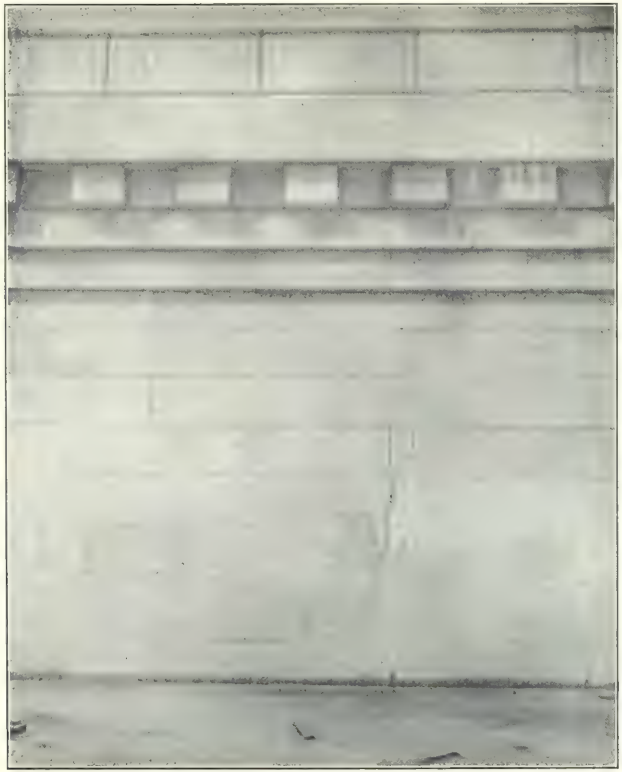


FIG. 1.—CRACK IN MASONRY OF CROSS RIVER DAM, AT STATION 14 + 89.

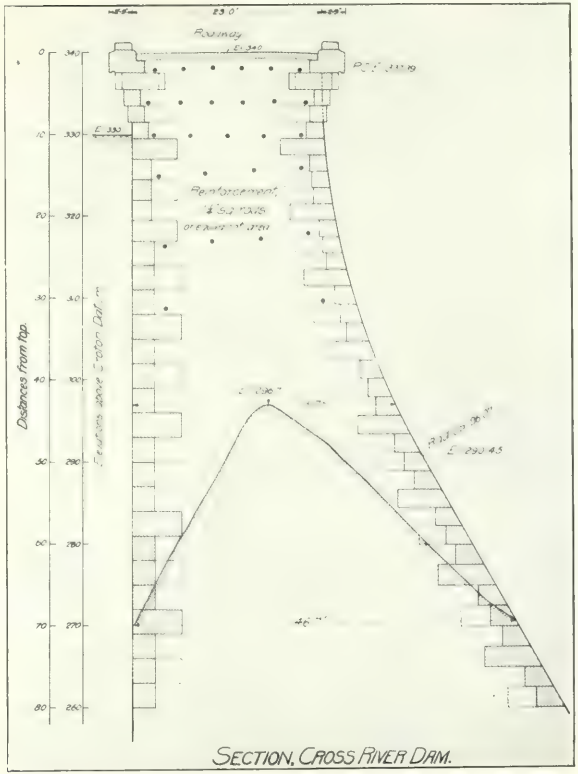


FIG. 2.—LIMIT OF CRACKS ON UP-STREAM AND DOWN-STREAM FACES OF DAM

masonry is indicated by Fig. 1, Plate XCVIII, and Fig. 2, Plate XCVIII, shows the completed structure as water was beginning to pass over the spillway, which, it should be noted, forms a part of the main dam. Mr. Merriman.

Atlas Portland cement was used exclusively in the construction of this dam. The masonry of the interior is "cyclopean," and is composed roughly of 50% of concrete and 50% of large stones. The concrete was mixed in the proportions of 1:3:6. All the stone used was a syenitic gneiss, and the sand was from local deposits, evidently of glacial origin.

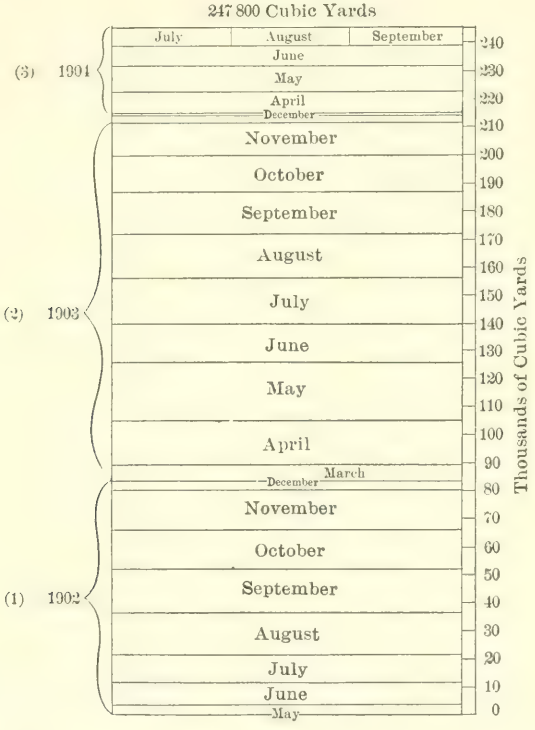


FIG. 1.

The temperature cracks which have occurred are shown on the profile, Plate XCVI. Those which occurred during the first winter (1903-1904) are indicated by solid lines, the upper ends of which show the height to which the masonry had advanced when work was suspended, in December, 1903. It will be observed that no cracks are shown between Stations 12 + 00 and 19 + 00. This is explainable by the fact that the upper layers of masonry between these stations were

Mr. Merriman. all put in during November and December of that winter. The weather during these months was extremely cold, and the surface of all the masonry then put in was frosted. The temperature range in this part of the work, therefore, was probably quite small, and the frosted condition of the surface also tended to prevent the discovery of any cracks which may have existed.

Before the winter of 1904-1905 arrived, the masonry of the dam had been completed, and the cracks shown by dotted lines made their appearance; the crack at Station 26 + 90 also again opened up.

During the third winter (1905-1906) no new cracks appeared, but all those which had formed during the second winter again opened up. A noticeable fact in connection with the re-opening of these cracks during the winter of 1905-1906 was that they did not all open up in the same ratio as during the previous winter; that is, one of the cracks was larger than a year ago, while one or both of its neighbors was smaller.

During the fourth and fifth winters (1906-1907 and 1907-1908) all the old cracks re-opened, though, as already noted, not always in the same ratio, and no new cracking could be observed.

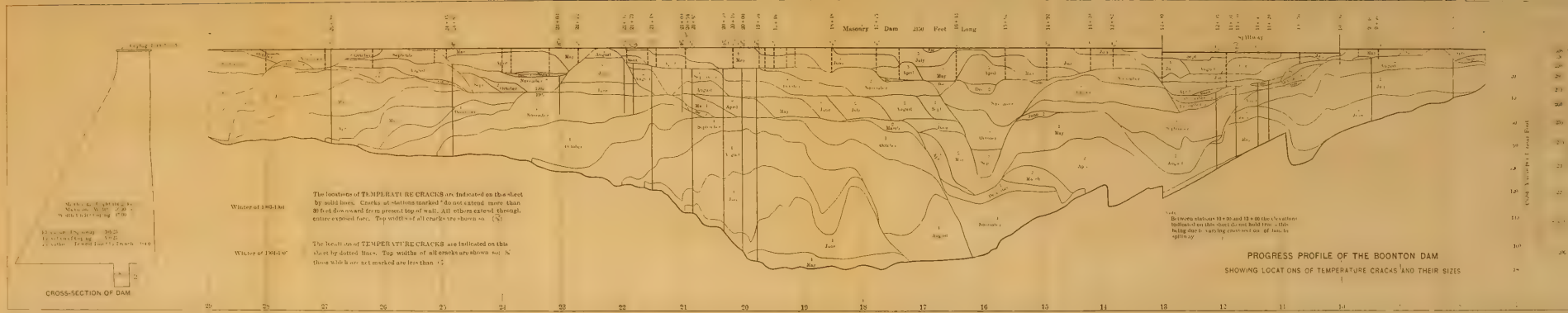
Seventeen main cracks have appeared in the masonry section of this dam, the designation, "main cracks," referring here, as elsewhere in this discussion, to cracks having a top width of $\frac{1}{16}$ in. or more. Besides these main cracks, there have appeared also 16 smaller ones, each having a top width of less than $\frac{1}{16}$ in., and, in addition to the 33 cracks which extend entirely across the top of the dam, there are about an equal number which do not extend across the section, but are confined to one-half or less of the crest width.

An approximate computation of the probable sum of all the cracks is as follows:

17 cracks, dimensions on Plate XCVI.	.....	2.50 in.
16 " averaging $\frac{1}{32}$ in.	.....	0.50 "
33 half cracks, " $\frac{1}{32}$ in.	.....	0.50 "
Total	.....	3.50 "

A general inspection of the profile, Plate XCVI, indicates that the main cracks occur at very regular intervals, and that this interval, except near the ends of the dam, is not far from 100 ft. It is also apparent that proportionately more cracks have developed in that portion of the dam in which the masonry was laid during the warmer months.

No very large quantity of water has passed through any except the main cracks. Through them, however, considerable seepage occurs. This seepage is greater in winter than in summer, and at some of them it is more or less visible during the entire year. In consequence of this seepage, the face of the dam is wet in the immediate vicinity of


Masonry Dam 1150 Feet Long
Spillway
CROSS-SECTION OF DAM
Winter of 1903-1904
Winter of 1904-1905
The locations of TEMPERATURE CRACKS are indicated on the sheet by solid lines. Cracks at stations marked "do not extend more than 30 feet downward from present top of wall. All others extend through entire exposed face. Top width of all cracks are shown so (¼).
The location of TEMPERATURE CRACKS are indicated on this sheet by dotted lines. Top widths of all cracks are shown so: ¼ those which are not marked are less than ¼.
Between stations 19+20 and 19+00 the elevations indicated on this chart do not hold true, this being due to varying crossection of foundation spillway
PROGRESS PROFILE OF THE BOONTON DAM
SHOWING LOCATIONS OF TEMPERATURE CRACKS AND THEIR SIZES

the crack through which it occurs, and in winter huge masses of ice are formed, so that a view of the face of the dam during the cold season presents a very interesting picture. The principal seepage occurs through the cracks at Stations 9 + 36 and 24 + 95. Mr. Merriman.

On March 17th, 1908, this seepage was measured and found to be 23 000 gal. per 24 hours.

In connection with his studies of the question of the temperature changes in masonry dams and their effects, the speaker visited the New Croton Dam on March 17th, 1906, and desires to record here his observations.

The crest of the New Croton Dam serves as a roadway, and is paved with rough concrete which was put down in two layers so as to form a horizontal expansion joint. This feature rendered the determination of the sizes of the cracks more difficult than at Boonton, where the top of the dam was finished off with a granolithic surface integral with the body of the dam.

For the purpose of removing the drainage from the crest of the New Croton Dam, a tunnel, parallel with its axis, and placed in the body of the dam below its crest, was provided. This tunnel afforded good opportunity for inspecting the cracks, which, however, here as well as elsewhere, had been caulked prior to the time of the speaker's visit.

There were found in this dam five large cracks. The largest of these measured about $\frac{1}{4}$ in., while each of the other four averaged about $\frac{1}{8}$ in. In addition to these larger cracks, there were a number of smaller ones, but just how many could not be determined, on account of the concrete roadway. These smaller cracks did not show in the drainage tunnel, but at the coping joints on the dam crest.

On the spillway section of the New Croton Dam the speaker found no very large cracks, though there were ten of small size, together with a number of others showing at the joints of the coping.

The general external conditions at this dam after its completion, therefore, did not appear to differ materially from those at Boonton, and, in general, they were also similar to those described by Mr. Honness as existing at the Cross River Dam.

It is not probable that the passage of water through temperature cracks can have any appreciable influence on the life of the structure; at any rate, no more than it has on the face joints of the up-stream side; moreover, most of the evidence at hand indicates in a general way that Portland cement does not deteriorate in the presence of water, but rather, and on the contrary, that it improves both in strength and durability as it becomes older. A greater and more important question seems to be, what may be expected to be the life of a masonry dam constructed with Portland cement as now manufactured?

Mr. Merriman. The passage of water through temperature cracks probably has no appreciable effect on the life of the structure, yet, by its presence on the visible face, it calls attention to the cracks themselves, and, to one unfamiliar with the technique of dam design and construction, such

LOCATIONS OF THERMOPHONES
IN THE BOONTON DAM.

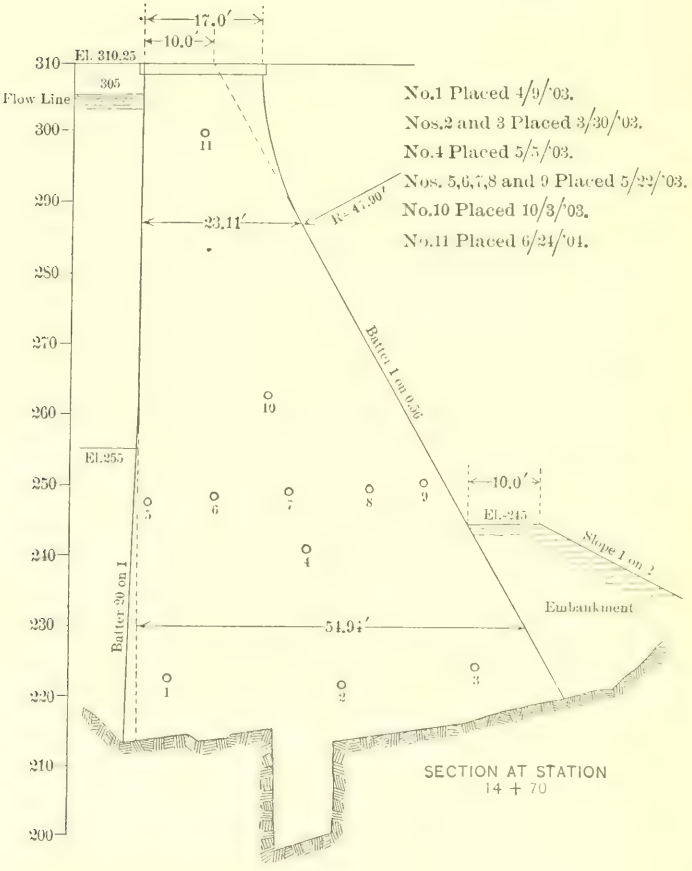


FIG. 2.

cracks appear only as indications of weakness, and no "explanation" can ever fully explain why such cracks should be permitted to exist.

The introduction of expansion joints, as referred to by the author, and as provided for in the plans of the Olive Bridge Dam of the Ashokan

Reservoir, for the new supply from the Catskill Mountains, is a step in the right direction, though experience alone will be able to tell whether all cracking can be prevented in so simple a manner. Mr. Merriman.

As work on the Boonton Dam progressed, eleven thermophones of the Whipple-Warren type were built into the masonry at Station 14 + 70 (Plate XCVI). The positions of the thermophones in the cross-section are shown by Fig. 2. These thermophones were designed to be sensitive and correct to within the nearest degree. During the first

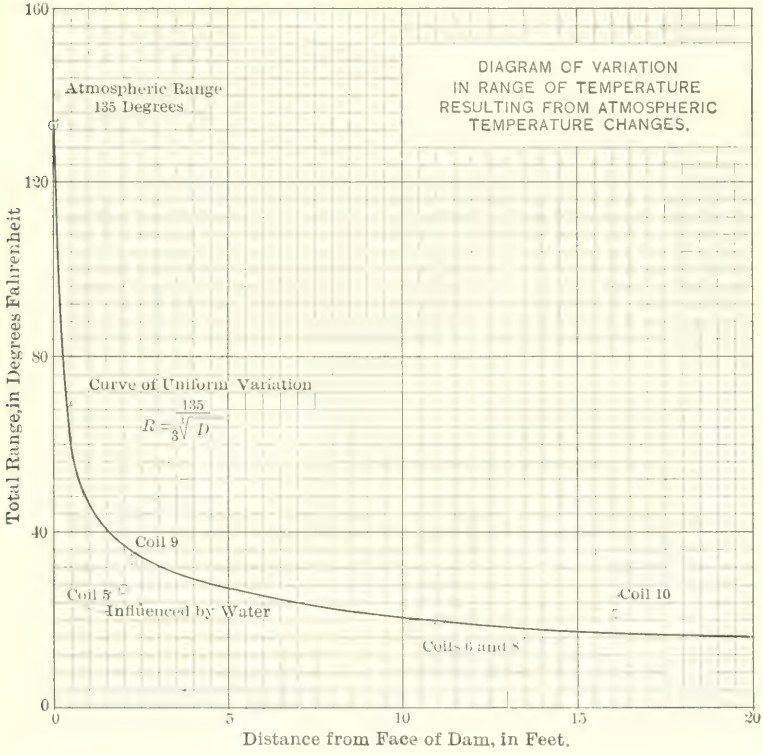


FIG. 3.

year the temperatures were read directly from the graduated scale of the resistance box, the point of equalization being determined by noting the cessation of vibration in a telephonic receiver when the circuit was rapidly made and broken. Later, the telephone arrangement was replaced by a galvanometer, the temperature being read, as before, on the graduated scale.

At various times trouble was experienced with the operation of the thermophones, as indicated by the sudden fluctuations in the temperature curves on Plate XCIX and Fig. 3. Then, later, the coils,

Mr. Merriman.

one after the other, without any apparent reason, ceased recording, until at this time none of the eleven is giving reliable results. An investigation into the cause of the trouble soon showed that it was due to the insulation of the cables leading to the coils. The cables were of the three-strand parallel variety, the three strands being brought together and then wrapped in a cotton armor, which was afterward treated with a water-proof coating. Each of the three strands consisted of a No. 12 copper wire, insulated by a layer of what appeared to be vulcanized black rubber. For some reason, whether on account of an excess of sulphur in the vulcanized rubber, or on account of the behavior of sulphur in the presence of Portland cement and its constituents, the sulphur in the insulation attacked the copper of the wires, and although perhaps not entirely destroying the wire, yet modified its resistance so that the results from the coils became valueless.

In future installations of this character, the speaker would recommend that each of the wires of the cables be insulated with three wrappings of silk and coated with paraffine after each wrapping. The three strands of which the cable is formed should then be brought together, wrapped twice with cotton and again coated with paraffin. The cable thus made should then be drawn into a lead armor, which armor should be soldered directly to the brass tube containing the thermophone coils. In this way the speaker believes that an absolutely moisture-proof lead can be obtained from the thermophone to a point outside of the masonry.

On Plate XCIX are plotted the records of the thermophone coils, together with the daily and monthly mean atmospheric temperatures. In order to avoid confusion, not all the records have been plotted, but only those showing the more continuous and uniform values. An inspection of the curves of these coils shows that all those placed prior to October, 1903, except Nos. 5 and 9, reached their minimum temperatures during the following April. These same coils reached their next minimum, however, during March, 1905, in spite of the fact that the atmospheric minimum occurred one month later than in the previous year. Coils Nos. 5 and 9, being but 2 ft. from the faces of the dam, reached their minimum temperatures not far from the time of the occurrence of the atmospheric minimum. The speaker, therefore, is led to the general conclusion that, owing to the heat generated by the cement during its setting, about one year must elapse before masonry in a dam of dimensions comparable with that at Boonton, will reach a temperature which will depend only on its position in the dam and on the changes in the atmospheric temperature. This conclusion holds true for all portions of a dam, since, if the heat generated by the setting action of the cement is being lost from any

FIG. 1.—DOWN-STREAM FACE OF BOONTON DAM, SHOWING ASHLAR FACING.

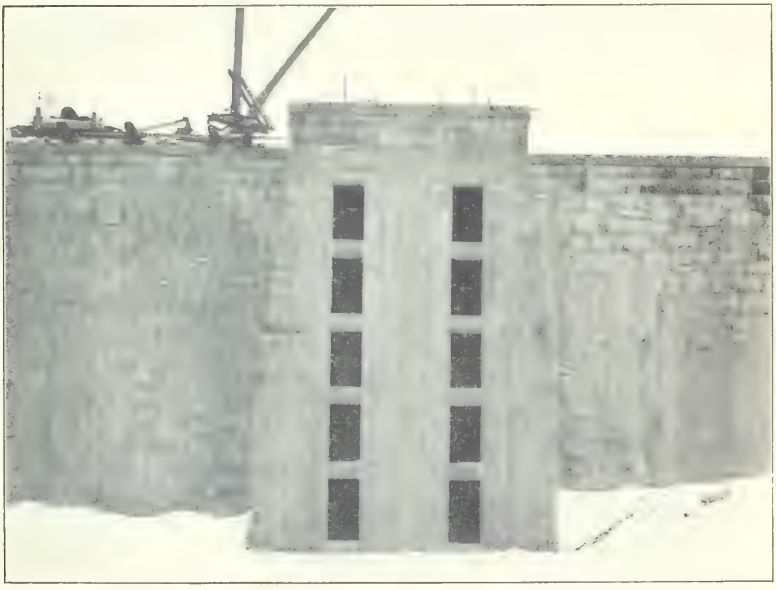


FIG. 2.—UP-STREAM FACE OF BOONTON DAM, SHOWING RUBBLE FACING AND GATE-HOUSE.

portion of the masonry, it must affect and tend to keep up the temperatures of all its parts. Mr. Merriman.

TABLE 6.—RESULTS OF THERMOPHONE OBSERVATIONS, BOONTON, N. J., DAM.

Coil No.	Distance from face, in feet.	Maximum temperature, in degrees, Fahrenheit.	Date of maximum temperature.	Minimum temperature, in degrees, Fahrenheit.	Date of minimum temperature.	Total temperature range, in degrees, Fahrenheit.
5	2.0	69	Aug., 1904	39	March, 1905	30
6	11.3	64	Sept., 1904	45	" 1905	19
8	11.0	63	Sept., 1904	43	" 1905	20
9	2.5	71	Aug., 1904	34	" 1905	37
10	16.0	68	Oct., 1904	46	" 1905	22

Referring again to Plate XCIX, it is easily seen that minimum and maximum temperatures near the center of the dam are reached approximately 30 days after the occurrence of the atmospheric minimum and maximum, respectively, and, moreover, the "lag" is seen to be proportional in some degree to the distance from the nearest face of the dam. Unfortunately, however, the observations do not seem to warrant the drawing of a curve of this "lag" or retardation.

Table 6 shows the principal results obtained from the thermophone observations on the Boonton Dam. It will be noted that no observations were used in the compilation of this table until the thermophones had been in place for practically a year. This was done in order to eliminate, if possible, the effect of the heat generated during setting, and heretofore referred to. The results shown in this table were then plotted on the diagram, Fig. 3, and by trial a curve most closely representing the observations was drawn. If R be the total range in temperature at any point in the mass, in degrees, Fahrenheit, and D be the distance in feet to the nearest face of the dam, then

$$R = \frac{135}{3\sqrt{D}}$$

where 135 is the total atmospheric range, as observed during the period from June, 1904, to March, 1905. During this period the minimum temperature reached was —18° fahr., and the speaker estimates that the maximum in the sun was 117° fahr., that in the shade being 98° fahr. This value, 135, is also probably not very far from the maximum atmospheric range ever attained in the latitude of New York.

Referring to the diagram, it is seen that the curve represented by this expression fits the plotted points very closely, and the speaker, therefore, has adopted it until future observations indicate that a change would be advisable. It must be noted, however, that near the

Mr. Merriman. minor limit the curve does not seem to fit in with the physics of the case, as it indicates that the range could never be zero, and that if the range were to be kept down to 1° fahr., the thickness of the masonry should be 182 250 ft. The speaker, therefore, prefers simply to state that the law expressed by this formula, for the present, at least, should not be considered as holding true for distances from the face of the masonry of more than 20 ft. or less than 0.5 ft.

It is well known that, after concrete and mortar have been mixed and placed into position, the temperature of the mass begins to rise. In a small cube this increase in temperature reaches a maximum about 18 hours after the cube is made, though, in a very large mass, this maximum may be delayed several days. The speaker would refer here to certain experiments made some years ago in the Lehigh University laboratory during which the temperature at the center of an 8-in. cube of 1:3 Portland cement mortar rose to 160° fahr. in 18 hours after the cube was made, the temperature of the laboratory being about 70° fahr. He would also refer to pages 414-416 of Johnson's "Materials of Construction," where it is stated that the temperature at the center of a mass of Portland cement concrete increased 93° fahr. in 7 days. The increase for the mortar cube was 90° fahr., and that for the concrete was 93° fahr., an agreement somewhat closer than one would anticipate, as the "heating" of cement would seem to depend largely on the materials of which it is composed and perhaps also on the process of its manufacture and mixing.

In the Boonton Dam nearly all the materials were deposited in comparatively thin layers, and the speaker, therefore, considers that their maximum temperatures were reached within 18 hours after placing. On Plate XCIX is shown the record of thermophone coil No. 11 by daily readings after it was put into position. It will be noted that, starting at about 77° fahr., this coil showed a rise of about 16° to 93° fahr. in about 13 days, and that about 11° of this rise occurred within 6 days. The facts of the placing of this coil are as follows: It was set into the wall on June 24th, 1904, and was immediately covered with about 12 in. of concrete. The temperature of the concrete at the time it was placed was 74.5° fahr. Seven days after placing the coil more masonry was placed above it, so that, within 14 days, it was buried to a total depth of about 11 ft. The increase in temperature recorded by this coil, therefore, was, it would seem, largely due to the heat from the masonry placed above it, and consequently it can be said that the temperature of the masonry placed above coil No. 11 and insulated from it by 12 in. of concrete already set, must have reached a temperature higher than 93° fahr.

On Fig. 4 are shown the results of hourly readings on thermophone No. 11. This coil was placed at 11.40 A. M., June 24th, 1904.

FIG. 1.—GENERAL CHARACTER OF INTERIOR MASONRY OF BOONTON DAM.

FIG. 2. COMPLETED BOONTON DAM WITH WATER PASSING OVER THE SPILLWAY

Readings were taken until 6.15 P. M., of that day, when they were dis- Mr. Merriman.
continued for the night. Next morning at 8 o'clock, in spite of a clear night, a minimum atmospheric temperature of 60° fahr., and notwithstanding an average atmospheric temperature of 68° fahr. from noon, June 24th, to noon, June 25th, the thermophone recorded a temperature of 87.5° fahr., a rise of nearly 10° since the previous evening. The curve of temperature for some hours after 8.00 A. M., June 25th, with a rising atmospheric temperature, showed a downward

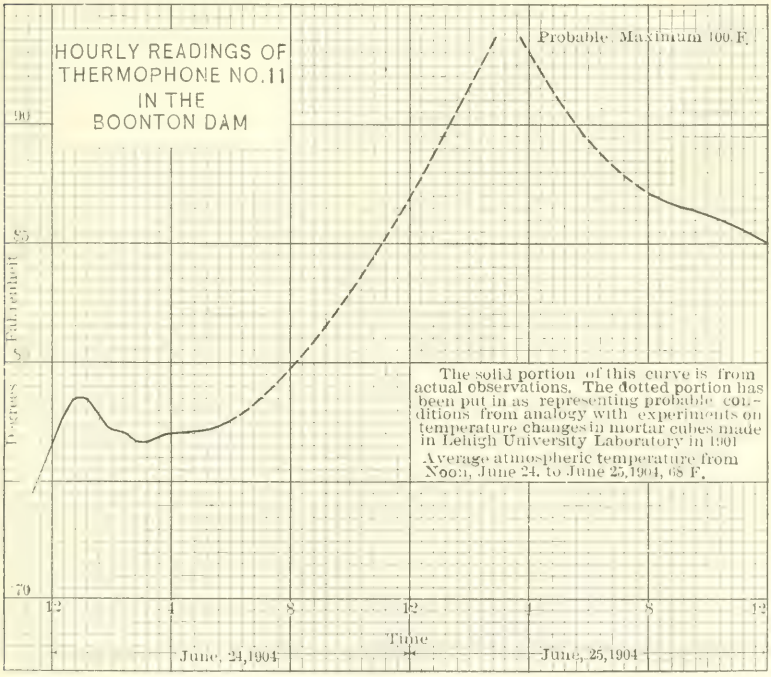


FIG. 4.

tendency, and this fact, together with the rise during the night and the general considerations of rise in masonry due to setting, as set forth in a previous paragraph, has led the speaker to indicate on Fig. 4 the probable curve of temperature for this coil between 6.00 P. M., June 24th, and 8.00 A. M., June 25th. The considerations which will explain the reasons for putting the probable maximum temperature reached at 100° fahr., are as follows:

The Boonton Dam is constructed of cyclopean masonry, composed roughly of 50% of large stones and 50% of concrete, having proportions of 1:3:6, therefore 25% of the mass is mortar and 75% stone,

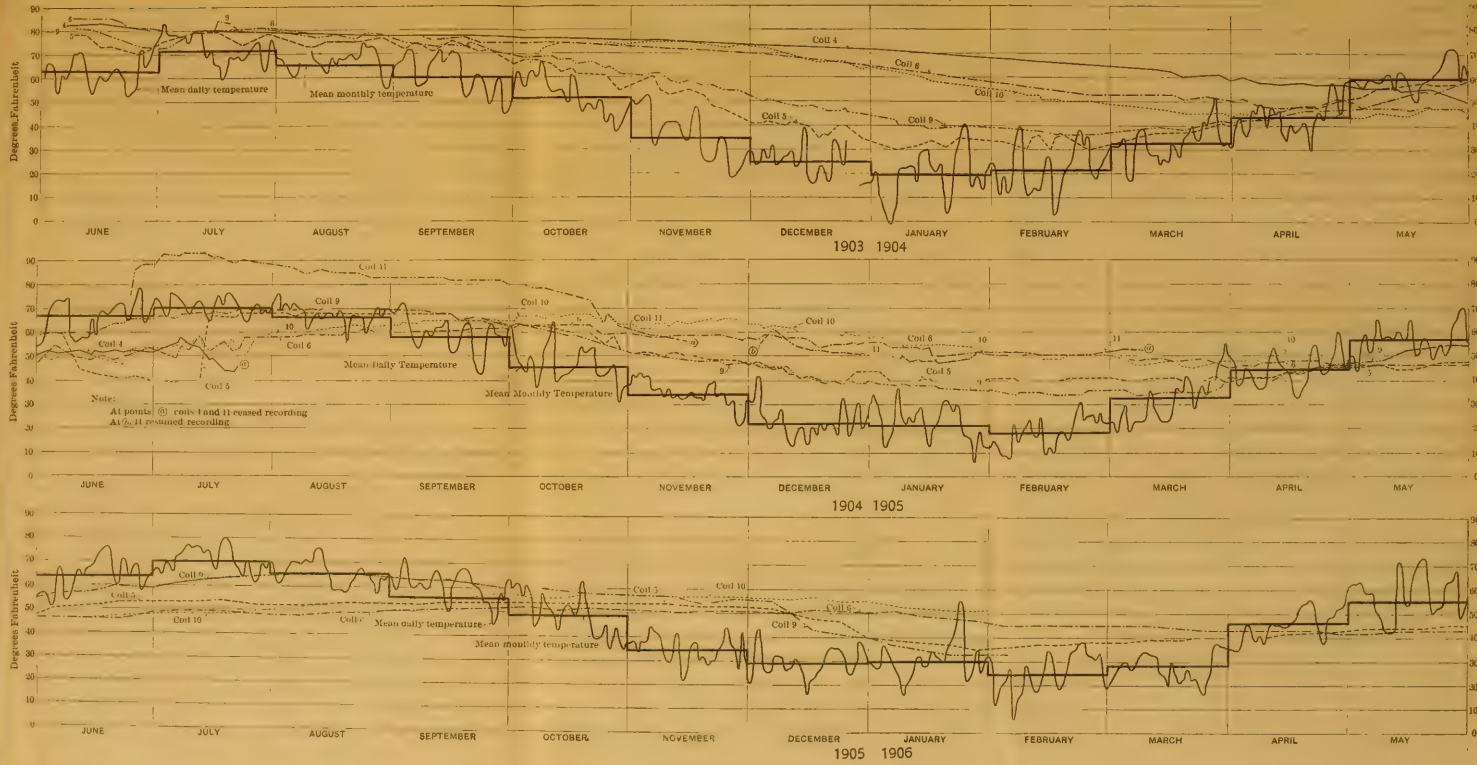
Mr. Merriman. whether large or small. If, now, it be assumed that the specific heat of the stone is 0.2 and that of the new mortar is 0.4, and that the temperature of the mortar alone would rise 90° fahr., it follows that the temperature of the mass will rise 36° fahr., and if the initial temperature of the mass was 74° fahr., that therefore the resultant temperature would be 100° fahr.

The average atmospheric temperature of the months during which masonry in a dam is usually placed is not far from 58° fahr., and if the chemical changes due to the setting of a cement are sufficient to induce in the masonry a rise of 36° fahr., then it is reasonable to suppose that the masonry in setting will reach a temperature of $58° + 36° = 94°$ fahr. This estimate, it will be noted, is based on the assumption that the average temperature of the sand and stone used in the construction will be the same as the average temperature of the atmosphere.

In arriving at a conclusion on this point the speaker has borne in mind the approximate character of the computations, and of the data entering them, and has preferred to state that the maximum average temperature which the masonry will attain shortly after being placed in position is 100° fahr., rather than the 94° fahr., above deduced. He has done so for the reason that it appears, from all considerations, to be true beyond a reasonable doubt that some of the masonry in a dam will reach this temperature, and that, therefore, it is conservative and on the side of safety to assert that all the masonry will do likewise. As far as observations of actual temperature cracks in a masonry dam tend to bear out this point, reference to Plate XCVI will show that all the main cracks which appeared on the top of the Boonton Dam are found in the masonry which was placed during the warmer months of the working year. The speaker, however, considers this only as a corroborative fact, as there are many other factors which may have a great influence on the location of the cracks. Among these should be mentioned the height of the masonry, the "racks" in which it was built, and the irregularity of the foundation profile.

Before leaving the question of the records of the thermophones, it will be interesting to indicate the difference in behavior between coils Nos. 5 and 9, coil No. 5 being within 2 ft. of the water face of the dam, while coil No. 9 is within 2.5 ft. of the exposed downstream face. During the winter of 1903-1904 the water in the reservoir rose just about level with coil No. 5. Both these coils reached their maximum temperature during August, 1904, No. 5 attaining 69° fahr., and No. 9, 71° fahr. No. 5, at this time, was not submerged by more than 20 ft. of water. During the following summer, however, No. 5 reached only 54° fahr., while No. 9 reached 65° fahr., No. 5 at this time being submerged about 55 ft.

TEMPERATURES OF THE INTERIOR OF THE BOONTON, N.J., DAM



Mr. Merriman.

The equalizing effect of the water in the reservoir is shown by the curves on Plate XCIX, which indicate that the water side of the dam is cooler in summer than the exposed face, and conversely. These curves also show that the temperatures of the two faces are equal twice each year, in December and April.

The setting of cement is the physical phenomenon resulting from chemical changes within its mass. By those who have made a study of the subject, these changes have been likened to the arrangement of the molecules into a definite and fixed form, resembling crystallization to some extent. It has been seen that the process of setting is accompanied by the evolution of heat, and Johnson* says:

"In fact, with quick-setting cements the temperature curve is a truer index of the setting period than the mechanical tests of firmness which are usually employed for this purpose."

Again, on page 416, he says:

"With slow-setting cements the rise of temperature cannot be observed with accuracy, and is smaller in amount than in case of quick-setting cements. In such cases the heat developed is dissipated because of its slow generation, and does, therefore, not become sensible to thermometric measurement."

From all points of view, therefore, it appears to be probable that Portland cement in setting assumes its final definite form and shape at a time when its setting temperature is a maximum. The speaker believes it to be an undisputed fact that the range of temperature which produces the maximum stress in a body, provided only that the body is not free to contract or expand, is the difference between the highest and lowest temperatures to which the body will be subjected. In view of all the evidence, therefore, he submits that in a masonry dam the maximum range in temperature which causes tensile stresses in the masonry is the difference between the temperature at which the cement assumes its set and the lowest temperature which may subsequently be reached. From this it at once follows, if the average setting temperature be 100° fahr., that the minimum total tensile stress producing range in any portion of any dam cannot be less than the difference between 100° fahr. and the yearly mean temperature of the atmosphere at the place where the dam is built. In the latitude of New York this minimum range, therefore, is 52° fahr.

Fig. 5 is a cross-section of the Boonton Dam on which have been plotted the curves of probable total temperature range within its mass, no attempt having been made to allow for the influence of the water in the reservoir, as the information at hand is too meager to warrant

* "The Materials of Construction," New York, 1906, p. 415.

Mr. Merriman. such a study. The problem, therefore, has been studied from the point of view of a simple masonry wall of the cross-section shown.

The atmospheric range of 135° fahr. is undoubtedly true on the exterior faces, but 2 ft. within the dam it is found that the application of the law of variation of the range in connection with the setting temperature of the masonry gives a range of only 72° fahr. It is also found that it is necessary to go 18 ft. farther into the dam to find a point where the range is 60° fahr. Fig. 5 is interesting in that it exhibits a condition of affairs in the center of a masonry dam almost diametrically opposed to that which has heretofore been believed to be true.

In connection with the measurements of the cracks in the Boonton Dam and the temperature ranges herein indicated, the speaker endeavored to obtain some idea of the tensile stress which the masonry actually carries. In order to do this it became necessary to make certain assumptions, as follows:

a.—The top width of a temperature crack is the same as its width at a point 2 ft. toward the interior of the dam. Were this not the case one might expect very shortly to see every concrete dam begin to "peel off." The crack is prevented from assuming the size indicated by the range in temperature at the surface by shearing forces which stress the masonry in directions parallel with the axis of the dam.

b.—That the coefficient of elasticity of the masonry is 2 500 000 lb. per sq. in.

c.—That the range in temperature which has produced the cracking and stresses is 72° fahr.

d.—That the coefficient of expansion of the masonry is 0.0000055 per degree, Fahrenheit.

The total visible cracking has been seen to aggregate 3.50 in., and the length of the masonry is 2 150 ft.

With these data it appears that, had no cracking occurred, the tensile stress in the masonry would have been 990 lb. per sq. in.; had the masonry been free to move, the contraction would have amounted to 10.20 in., and that since an elongation of 6.70 in. remains unaccounted for, the tensile stress in the masonry must now be about 650 lb. per sq. in., a result which agrees fairly well with what might be supposed to be the tensile strength of a large mass of masonry if the strength of small test pieces of its mortar at one year showed by test 380 lb. per sq. in.

If tensile stresses exist in a masonry dam and act on planes at right angles to its axis, there is no admissible reason for believing

Mr. Merriman.

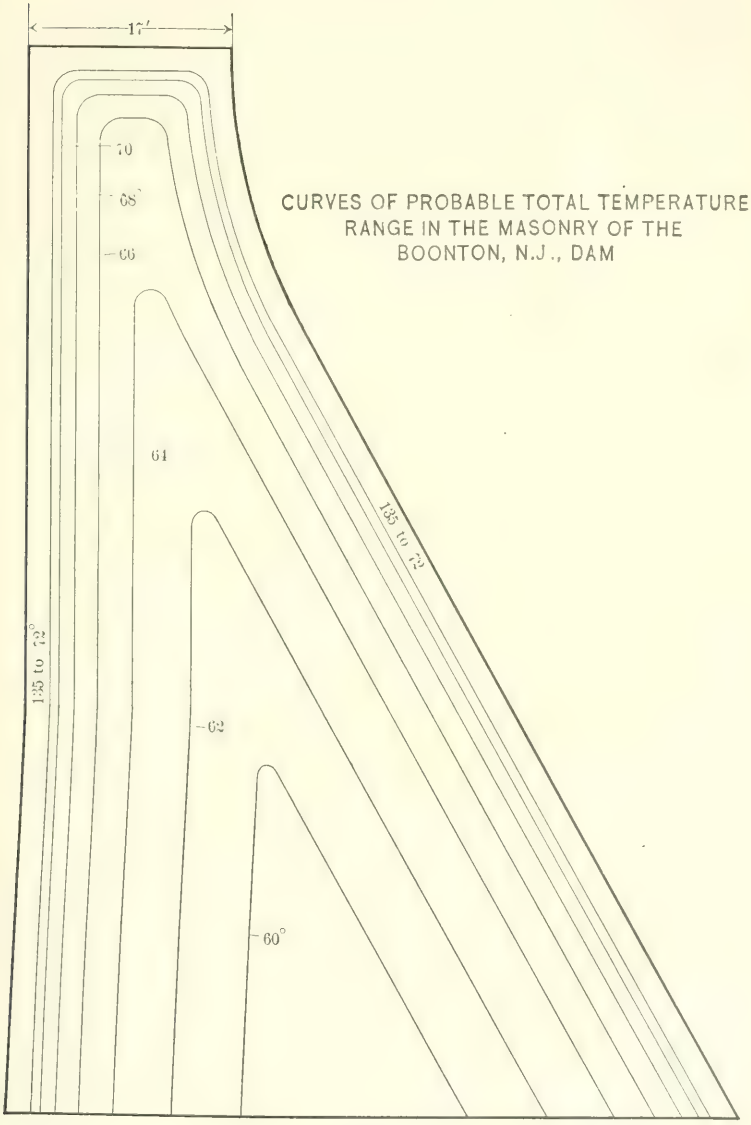


FIG. 5.

Mr. Merriman. that such stresses do not exist and act with equal intensity on all planes and at every angle. This being the case, the stresses due to this cause should not be overlooked when the design of a masonry dam is under consideration, particularly when it is remembered that the forces which act on a dam cause tensile stresses within its mass.

An entirely independent confirmation of these results has come to hand in the interesting facts presented by Mr. Honness, who has pointed out that the cracking in the Cross River Dam probably did not extend entirely through its cross-section because the leakage, when the water stood below a point 46 ft. below the dam crest, was very slight. It is probable, therefore, that the cracks did not extend much more than 46 ft. below the dam crest, and it would seem to follow that at this point the temperature range was just sufficient to overcome the tensile strength of the masonry. Referring to Fig. 5 it will be seen that the temperature range at a point in the center of the dam and 46 ft. below its crest would be not far from 61° fahr., the cross-sections of the two structures being very similar. This range in temperature, assuming the same constants as previously used, would indicate that the masonry had a tensile resistance of about 800 lb. per sq. in., which agrees very well with that determined for the Boonton Dam when all the differences are considered, and when it is remembered that this dam was not a year old when the cracks were observed. Its interior temperature, therefore, was higher than it will be later, and consequently the actual range in temperature was less than that used in the computations. Such a condition would make the apparent strength of the masonry somewhat higher than its true value. The steel rods used in the upper portion of this dam may also, possibly, have had some influence on the size and positions of the cracks, though the speaker does not feel that the introduction of steel into a masonry mass can have any other action than to prevent the localization of cracking. The total cracking due to temperature changes must be the same, whether or not steel is used.

The theoretical deductions thus made on a large masonry mass seem to be fairly well confirmed by the physical facts of its behavior after visible surface cracking has occurred. Yet it is by no means certain that they are more than appproximately correct, not only on account of the uncertain nature of the data, but also on account of the many factors entering the problem for which no allowance in the present state of knowledge can be made.

In addition to the thermophone observations at Boonton, the attempt was made to ascertain whether or not there was any change in length between the ends of the masonry structure due to seasonal temperature changes. For this purpose lead plugs were inserted near each end of the masonry and about 2 000 ft. apart. At right angles with the axis of the dam and on a line with each of these plugs two heavy

posts were set into the ground penetrating well below the frost line. Using a transit, a point was then put on each of them in line with a point on the lead plug. Frequent observations with the transit failed to detect any shortening movement; such motion as was detected may have been due to errors of observation, and the information gained was negative only. Mr. Merriman.

In concluding his discussion of the Boonton results, the speaker would summarize as follows:

1.—About 12 months must elapse after the placing of masonry in the cross-section of a dam comparable with that at Boonton before the heat generated by the chemical changes incident to the setting of the cement becomes lost to an extent sufficient to render its existence a negligible quantity.

2.—After the expiration of 12 months, minimum and maximum temperatures near the center of the masonry are reached approximately 30 days after the occurrence, respectively, of the seasonal atmospheric minimum and maximum.

3.—The range in temperature at any point in the interior of a masonry dam, due to external temperature changes, is dependent on the exterior atmospheric temperature range, and on the distance of the point from the face of the dam. For distances up to 20 ft. and perhaps for greater ones, the interior range varies inversely nearly as the cube root of the distance from the face.

4.—Immediately after concrete and mortar are placed in a dam, the temperature of their mass rises and probably reaches a maximum in about 18 hours after the placing. This maximum is about 100° fahr. for cyclopean masonry.

5.—It may be inferred that the constituents of the mortar and concrete assume their final form and shape at the time when their maximum temperature is reached. This being the case, it would appear that nearly all portions of a masonry dam are under a tensile stress, practically from the time the materials composing it are placed in their position, and consequently it seems to follow that the stress producing range in temperature is more nearly equal for all portions of the structure except those very close to the exterior of the mass than has hitherto been supposed. This deduction follows from the consideration that, if the maximum temperature attained is 100° fahr., then the stress producing range is the difference between 100° fahr. and the lowest temperature subsequently attained.

6.—Approximate computations indicate that the ultimate tensile strength of the mass of a masonry dam constructed of Portland cement is not far from 700 lb. per sq. in.

7.—From the evidence at hand, it would seem that the use of a slow-setting cement would tend to keep down the setting temperature, and, therefore, it might be expected that a structure made of such a cement

Mr. Merriman. would show less cracking than one made of a cement which takes its set more rapidly.

In Table 4 the author gives the coefficient of expansion for masonry in large masses. This coefficient of expansion, 0.00000307, is somewhat smaller than that which has been used heretofore in the design of masonry structures, in fact, about one-half, and the speaker would suggest that this difference may be accounted for, at least, in part, by the fact that, in deducing his value, the author used the range in atmospheric temperature, whereas the temperature range which was effective in the mass of masonry was materially less.

The present-day design of massive masonry structures is deficient in so far as a proper knowledge is had of, and proper allowance is made for, the effect of the tensile stresses due to temperature changes. These stresses, when taken together with the other tensile stresses resulting from the forces acting on the mass, may give rise to resultant stresses in excess of conservative practice; therefore, all facts bearing on this question are of value, and the author is to be congratulated for having presented in such concise form the results of his extended observations on the highest masonry dam in the world.

Mr. Brown. WILLIAM LOWE BROWN, M. AM. SOC. C. E.—As a short account of the cracks which appeared in the Assouan Dam may be of interest, the speaker will describe them to the best of his recollection.

Mr. Gowen's paper gives a very clear account of the cracks which appeared in the New Croton Dam, and of the very careful measurements which the author had taken for so long a period. Such painstaking work is always of great value to others, and engineers should be grateful to him for what he has done.

Mr. Gowen concludes by suggesting that, in order to avoid cracks, certain precautions should be taken; these may be summarized as follows:

(*a*) Sudden changes of section should be avoided.
(*b*) The work should be carried out in horizontal layers, and all racking and stepping avoided.
(*c*) The work should not be carried on in hot weather.

In building the Assouan Dam, owing to unavoidable conditions, these rules were violated in a very flagrant manner, especially in the lower half of the dam, as will be seen from the following:

(*a*) It was impossible to avoid sudden changes of section, for provision had to be made for allowing the River Nile when in flood to pass without damaging the structure, and this could only be done conveniently by leaving sluices through the body of the dam. There were 180 of these sluices, 120 of which were 23 ft. high and 6½ ft. wide, and the remainder 12 ft. high and 6½ ft. wide. In addition to these, there were buttresses between each set of ten. The greatest variation in

section, however, was caused by the very irregular nature of the foundation line, which was cut up by five distinct river channels, divided by high points which were only flooded at high Nile; and the irregularity was increased still further by the fact that the rock at the bottom of the various channels was usually much softer than that between them. Consequently, in order to get a solid rock foundation, the excavation in the channels had to be carried to a greater depth below the original rock line than in the higher portions; so great, in fact, was this sudden drop at one point, that it was deemed advisable to cut away the good rock at the sides in a series of steps in order to make it less sudden. Mr. Brown.

(*b*) It was impossible to carry up the lower portion of the work in horizontal layers, for only two of the river channels could be blocked in any one year, and it was necessary to complete the excavation and carry up the masonry above the mean river level in these channels in one working season; the result was that, in the earlier stages, each season's work consisted of a large mass of masonry in the newly exposed foundation and a straggling piece of work above that of the previous year, and the first section was almost completed before the last part of the foundation was commenced.

(*c*) It was impossible to avoid carrying on the work in the very hottest weather, for the working season was only about eight months—December to August—and as, during the first four of these months, most of the time was spent in excavating the foundation, it was necessary to do the bulk of the work, in the lower portions of the dam, in May, June, and July, during which months the mean temperature remained above 90° fahr. The maximum midday shade temperature was very frequently 110° fahr., and occasionally 122° fahr.; in the sun it was very much hotter (170° fahr., or more). The minimum winter temperature is usually about 50° fahr. and occasionally it goes down to 40° fahr.

Being built under such conditions, it would be reasonable to suppose that the Assouan Dam would be very badly cracked. This, however, was not the case. There were perhaps six or seven cracks, altogether, but none of them was serious. The largest crack occurred at one of the most sudden changes of section, due to a high step in the foundation, at the side of a sluice and in close proximity to one of the buttresses. This crack extended from the top of the dam almost down to the rock foundation, and was about $\frac{1}{4}$ in. wide at the foundation and rather more at the top, in cold weather. It extended through the dam from side to side.

Calculations show that in a dam of this length—6 500 ft.—the total amount of contraction would be about 1 ft., assuming the range of temperature of the masonry to be only 30° fahr. In point of fact, the aggregate width of all the cracks could not have been more than from 1 to 2 in. The masonry, however, must have contracted by an amount

Mr. Brown. commensurable with that given by the calculation for the stress set up by the tendency to shrink. Had there been no cracks, this would have been 375 lb. per sq. in., if the modulus of elasticity were 2 500 000 lb. per sq. in., and if it were higher, which is much more likely, the stress would be greater. Even if the masonry remained under a tension of 200 lb. per sq. in., the aggregate width of the cracks must have amounted to about 6 in.; it is probable, therefore, that there were a great number of microscopic cracks which were not visible.

It has frequently been observed that in a concrete structure there are more cracks than in a similar structure of stone masonry. This is probably because concrete can only relieve itself of stress by a fracture of the whole mass, while a masonry structure can give way by creeping and shearing between the component stones and their beds.

The amount of leakage through these cracks was very small, and, at the worst one, only amounted to about 5 or 10 gal. per hour, and decreased very considerably after the reservoir had been full for some time. The small leakage compared with the size of the cracks is usually assumed to be due to the irregularity of the fracture, and to the fact that the width is much less in the heart of the masonry, where the temperature changes are much smaller; this is probably the correct explanation.

A reason for the decrease in the leakage, after the reservoir has been full for some time, has suggested itself to the speaker; it is that the masonry on the up-stream face swells when it becomes wet, and in so doing closes up the crack.

That swelling is a very real factor was clearly illustrated in the observations, referred to by Mr. Gowen, which were made by Sir Alexander Binnie on a concrete beam 100 ft. long and 1 ft. square in section. The changes of length due to changes in the humidity of the concrete frequently almost entirely masked those due to changes of temperature, and it was only after arrangements had been made for measuring the amount of moisture in the concrete, every time an observation of the length of the beam was made, that a standard of comparison could be established for the various observations of length, and that the changes of length due to changes of temperature could be distinguished from those due to changes of humidity.

With regard to Mr. Gowen's paper, there are two points which might be mentioned. The addition of a foundation line to the longitudinal view of the dam would tend to show more completely the changes of section from point to point, and might explain the position of the cracks.

The probable reason why the coefficient of expansion of the masonry as found by Mr. Gowen is so small has been pointed out by Mr. Merriman. It is certain that the range of temperature of the masonry itself must have been less than that of the atmosphere, and, had it been

possible to observe the former instead of the latter, a very accurate value for the coefficient might have been obtained. That this would have been the same as that shown in Table 4 is not very probable, for there is no reason why various kinds of concrete and masonry differing so greatly in their component materials should show any invariability in their coefficients of expansion. Mr. Brown.

There is one sentence in Mr. Gowen's paper to which the speaker wishes to take exception. After stating that the coefficients he obtained may have been affected by the gradual drying out of the masonry, he continues: "Possibly the higher coefficients (the first two in Table 4) may be due in a measure to this." The first value of the coefficient is obtained from the increase of length due to a rising temperature, and therefore any shrinkage due to drying would have tended to reduce the coefficient and not to increase it.

Mr. Merriman has stated that, in the use of the thermophones on the Boonton Dam, there was great difficulty with the insulation of the wires. He suggests that the wires be incased in lead. This was done in the case of the resistance thermometer built into the Assouan Dam, and there was no difficulty in getting accurate temperatures.

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PAPERS AND DISCUSSIONS

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RAIN AND RUN-OFF NEAR SAN FRANCISCO, CALIFORNIA.

Discussion.*

BY MESSRS. E. C. MURPHY, EDWIN DURYEA, JR., H. L. HAEHL, AND A. C. TOLL.

Mr. Murphy.

E. C. MURPHY, ASSOC. M. AM. SOC. C. E. (by letter).—In this paper the author has given the results of his study of rain distribution over an area of about 5 000 sq. miles lying to the southeast of San Francisco; also, the results of computations of rain and run-off from three small basins in this area. As the rain was measured at 56 places on this area, for periods varying in length from a few years to 54 years, and as the run-off from the three basins was carefully measured for several years, these results are of more than ordinary value.

The normal rain distribution over this area for the 54 years, 1849 to 1903, is shown by isohyetals. Comparing these lines with the lines for this area for the period 1870 to 1901, prepared by the United States Weather Bureau, one finds comparatively little similarity. The number of points where rain is measured is comparatively small, and many of these are in cities at low elevation. Much of the uninhabited country at high elevation is without rain measurement. It is necessary, therefore, in drawing these lines, to supplement the precipitation and elevation data with judgment based on general principles and a personal knowledge of the country. All the rain records do not begin

* This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., printed in *Proceedings* for April, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

at the same date and in some of them there are missing months and years. The computed normal at a place depends to some extent on the method used to interpolate the missing data. It is to be expected, therefore, that the location of isohyetal lines by different persons will differ somewhat. Mr. Murphy

From the isohyetals for each year the mean depth of precipitation over a drainage basin is computed. From them, also, the number of rain gauges necessary in a basin to give a required degree of precision of rain measurement is seen. From them, too, the change in distribution at a given place from year to year is seen.

The author gives a run-off curve for the Coast Range based on his study of the rain and run-off from three small basins. One would naturally expect the data from an area of less than 50 sq. miles, on which the rain and run-off were carefully measured, to be concordant and give a well-defined run-off curve. Such is not true for this case, however. It is seen that all the points representing the data of the Upper Crystal Springs Basin fall to the right of the author's run-off curve, and five of the seven points of the Pilarcitos and San Andreas Basins fall to the left of this curve. It is clear that each of these three basins requires a run-off curve of its own, if the computed run-off is to have a reasonable degree of accuracy.

The third column of Table 5 gives the measured run-off from the Upper Crystal Springs Basin, and the fourth column gives the run-off taken from this curve. The latter, for the 11-year record, is 88% greater than the former.

TABLE 5.—RAIN AND RUN-OFF FROM UPPER CRYSTAL SPRINGS BASIN.

Year.	Rain, in inches.	Run-off, in inches.	Run-off from curve, in inches.
1877–78	51.5	13.8	18.0
78–79	37.4	0.0	8.5
79–80	40.7	1.1	10.2
80–81	36.9	0.0	8.0
81–82	23.6	1.3	2.6
82–83	22.9	1.4	2.6
83–84	38.0	4.3	8.0
84–85	26.5	1.8	3.6
85–86	37.3	9.5	8.5
86–87	26.0	3.1	3.1
87–88	25.4	4.2	3.0
	Totals.....	40.5	76.1

These data appear to show that run-off curves may differ by a large amount, even for small adjoining basins which appear to be similar.

The author's illustration, Fig. 2, of the necessity of treating drain-

Mr. Murphy.

age basins in subdivisions, in computing the depth of rain, is apt. It shows the need of a large number of precipitation stations in a basin.

The data of this paper show clearly that run-off computed from precipitation is subject to very large errors.

Mr. Duryea.

EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—Mr. Grunsky should be thanked for putting the results of his long investigation of rainfall and stream flow near San Francisco on record for the use of other engineers. During the past five years and more the writer has spent most of his time on similar investigations—studies of the rainfall and stream flow of Coast Range and Sierra areas—with a view to the utilization of the streams for water supply, power, or irrigation. His work has included, not only much study and deduction in the office, both from published records and original records secured in the field, but also the establishment of more than 130 rain gauges and about 20 stream-gauging stations. Therefore, he has accumulated a large number of engineering data on the subject of California rainfall and stream flow, and feels that he can add something to the value of Mr. Grunsky's paper by discussing it. However, as urgent work prevents putting these data into shape for publication at present, the writer is obliged to restrict his discussion of Mr. Grunsky's paper to comments on some statements from which he differs, hoping to present his own rainfall and stream-flow studies for publication somewhat later.

Referring to Mr. Grunsky's high-mountain or Sierra Nevada run-off curve, the writer has checked its correctness quite exhaustively, and finds it to be, as representing a nearly minimum run-off, quite an accurate representation of Sierra conditions. He cannot agree with Mr. Grunsky, however, that it represents in any way mean values of Sierra run-off. Independently, the writer has worked up the rainfall and run-off relations of the King, Tuolumne, Stanislaus, Mokelumne, Cosumnes and American Rivers for all rainfall years (September to August, inclusive) previous to, and including, 1906-07, making use of all published and many unpublished data, and of isohyetose lines and numerous base rainfall stations, in order to deduce reasonably accurate average rainfalls for each area gauged and for each year. The results of this study, expressed as points plotted on rectangular co-ordinates, representing rainfall and run-off, respectively, are comprised in 79 points showing the rainfall and its resulting stream flow; and, of the 79 points, only 9 fall appreciably below Mr. Grunsky's curve. Of these 9 points, 5 are for rainfalls of more than 50 in., or for years of such high rainfall that they are of little interest as affecting the reliability of a water supply, and 4 are for rainfalls between 45 and 50 in. For those years in which the rainfall is less than 45 in.—the only years of practical interest in connection with the use of Sierra streams—Mr. Grunsky's curve represents the worst Sierra run-off

conditions of which any record has yet been secured, and not mean conditions, as he states. Mr. Duryea.

In plotting his curves with respect to the average rainfalls and stream flows of groups of several years, instead of the rainfalls and stream flows of each separate year (rainfall season), Mr. Grunsky has departed from the usual meaning given to run-off curves by engineers, and, in the writer's opinion, the value and general applicability of his curves have been much lessened by the change. In nearly all investigations for city water supplies (and in a somewhat less degree for power and irrigation projects), the only years of much interest are the two or three forming the period of least run-off or stream flow, generally coincident with the period of least rainfall; and the average water production for several years is not usually of much practical interest. The Spring Valley Water Company's Peninsular system, however, is an exception to this general rule, as the small area furnishing the supply has made it necessary to develop so great a storage that the floods of the best years are utilized by storing the water, even for ten years, until it is needed in the bad years. The Crystal Springs Reservoir, which has been in use nearly 20 years, has been filled but twice, and its dam is as yet uncompleted and still 20 ft. below its proposed ultimate height. It is doubtful if there is any other large system with such excessive storage; and rates of storage and of daily supply per square mile derived from this area are of little value for general application elsewhere. While Mr. Grunsky's method of plotting his Coast Range curve, from means instead of from yearly run-offs, may apply well enough to the fully-developed area from which the data are derived, this could have been achieved as well by a curve plotted from the actual yearly run-offs, and the data would then have been more safely applicable to similar areas as yet undeveloped. The danger in plotting mean instead of yearly run-offs lies in the minimizing effect thus apparently given to the bad influences of the years of least run-off. To the writer, there seems to be no justification for applying the average run-off method to the Sierra streams, as yet undeveloped.

It is to be regretted that Mr. Grunsky's two run-off curves are shown on one diagram, as the mingling of the symbols relating to the Sierra curve with those relating to the Coast Range curve makes it impossible to study intelligently the relation of each curve to its respective data. The writer has found it necessary to use a separate diagram for each run-off curve projected.

Mr. Grunsky's statement that entering his run-off curve with the average rainfall from a large area of widely varying rainfalls will give too low a run-off is true, theoretically, but the magnitude of the error is dependent on the shape and rainfall characteristics of each area, and is generally very small. Table 6 shows this error for six

Mr. Duryea.

Sierra rivers, the rainfalls considered being the mean rainfalls as shown by isohyetose lines:

TABLE 6.

River and gauging station.	Area, in square miles.	Run-offs, by Grunsky's Curve, from: Average rainfall of whole area.	Run-offs, by Grunsky's Curve, from: Rainfall of parts between 10-in. isohyetose lines.	Increase in average rainfall, corresponding, by Grunsky's curve, to the larger run-offs.
King River, above Sangers	1 742	8.1 in.	8.6 in. = 106%	3.3%
Tuolumne River, above La Grange	1 557	19.7 "	20.3 " = 103%	1.7%
Stanislaus River, above Knight's Ferry	935	20.2 "	21.0 " = 104%	1.9%
Mokelumne River, above Clements	639	19.2 "	20.1 " = 105%	3.0%
Cosumnes River, above Live Oak Suspension Bridge	580	20.7 "	21.4 " = 103%	1.7%
American River, above Fair Oaks	1 925	26.4 "	27.2 " = 103%	1.5%

The mean rainfalls on the King River area vary from about 9 in. in the valley to 44 in. in the high mountains, those of the Tuolumne River from about 16 to 57 in., the Stanislaus from 18 to 58 in., the Mokelumne from 20 to 60 in., the Cosumnes from 26 to perhaps 63 in., and the American from about 25 to perhaps 65 in.; yet the error in run-off from entering the Grunsky curve with the average mean rainfall of each entire area is only from 3 to 6%, with an average error of only 4% for the six streams. If the average mean rainfalls of the entire areas are increased by from 1.5 to 3.3%, with an average increase of 2.2%, the run-offs obtained from the curve will then equal those resulting from dividing up each area into from five to nine parts and getting the run-off of each part separately. Because of the small error shown for these six streams and also because all but minimum run-offs are represented as well by percentages of the rainfall as by curves, it seems that only in very unusual instances will the treatment of large areas as a whole cause errors in the resulting run-off of any importance. It is probable that the streams rising on the east slopes of the Sierra Nevadas and flowing into the Great Basin are exceptions to this general rule.

Messrs. Haehl and Toll.

H. L. Haehl, Assoc. M. Am. Soc. C. E., and Asahel C. Toll, Jun. Am. Soc. C. E. (by letter).—The difficulty of determining the mean rainfall for any considerable area in the mountain water-sheds of California is well recognized by hydraulic engineers having to deal with water-supply or power development within the State. This difficulty, due in large measure to the great variations in rainfall conditions within even very small areas, is accentuated by the fact that the longest and most reliable rainfall records are those from valley stations, so that all local records have to be amended by comparision with these

longer records before reliable mean seasonal rainfalls can be deduced for the mountain stations, and even then it remains to decide how much territory is represented by each station. Messrs. Haehl and Toll.

A valuable aid in this determination is an isohyetose or rainfall distribution map such as that presented by the author. Such a map, based on all available data, presents in graphic form all known facts of the distribution of rainfall over the area covered, and affords a ready means of judging the mean seasonal rainfall at any point or for any included area. The writers, having access to many data not available to the author at the time of his investigation, have worked out, for the territory covered by those data, a revised isohyetose map; and they present herewith the results of their studies as an additional contribution to the knowledge upon this subject now made available to the profession.

The records here used are largely the results of recommendations made in 1902 by F. S. Washburn, M. Am. Soc. C. E., then Consulting Engineer to the Bay Cities Water Company, the observations having been made within the catchment areas of that company. The collection of these and other data, covering evaporation, humidity, and stream flow, in Santa Clara County, as well as the other engineering activities of that company, was under the direction of Edwin Duryea, Jr., M. Am. Soc. C. E., who became, and for several years continued to be, its Chief Engineer. The field work was under the direct supervision of H. L. Haehl and K. F. Cooper, Associate Members, Am. Soc. C. E., as Assistant Engineers. Unfortunately, most of the final tabulations of these data, together with the various studies in which they were used, were burned in the San Francisco offices of the company during the Earthquake-fire of April, 1906, and the work of collecting and re-compiling the large mass of information, preserved in the field offices in the original field books and notes, has not since been undertaken in full. It is hoped that the records covering run-off, evaporation, etc., in Santa Clara County, which are unusually complete and extensive, may at some time form the basis for a paper before this Society, but, owing to the lack of time for putting those data into presentable form, the present discussion will be confined principally to the question of rainfall distribution.

The collection of rainfall records was begun in December, 1902, when 24 rain gauges were located at various representative points in the 193 sq. miles of catchment area of the Coyote River. In September, 1903, at the beginning of the rainfall season, 1903-04, 24 additional rain gauges were set on the Coyote catchment area: 22 on the Uvas and Llagas Creeks, representing 53 sq. miles of catchment area: and 34 on the Arroyo Honda and Arroyo Valle catchment areas, of 88 sq. miles. Later, several additional rain gauges were located at various points in these areas, bringing the number of rainfall stations main-

Messrs. Haehl and Toll.

tained by the company up to more than 100; and with the stations maintained by the U. S. Weather Bureau, the Southern Pacific Railway, Lick Observatory, and a few individuals, making a total of 122 record stations within the area of 1 000 sq. miles shown on Plate C, or about 1 to each 8 sq. miles. Most of the company's records were discontinued in 1905, but some have been brought up to date and will probably be continued indefinitely. The periods of record of the various gauges are given in Tables 8 to 13, which deal with their records and the deductions therefrom, treating them in groups according to their location and the similarity of their rainfall characteristics to those of certain nearby stations, the longer-period records of which were available for use in this study.

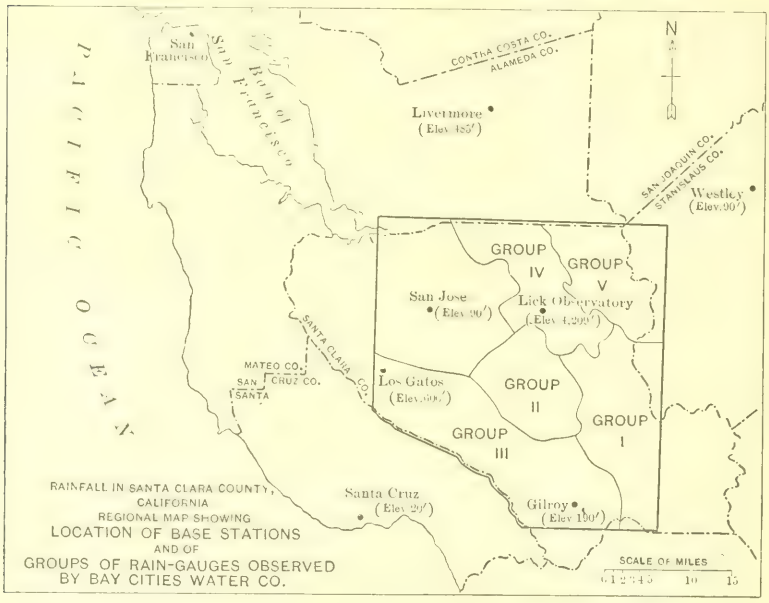
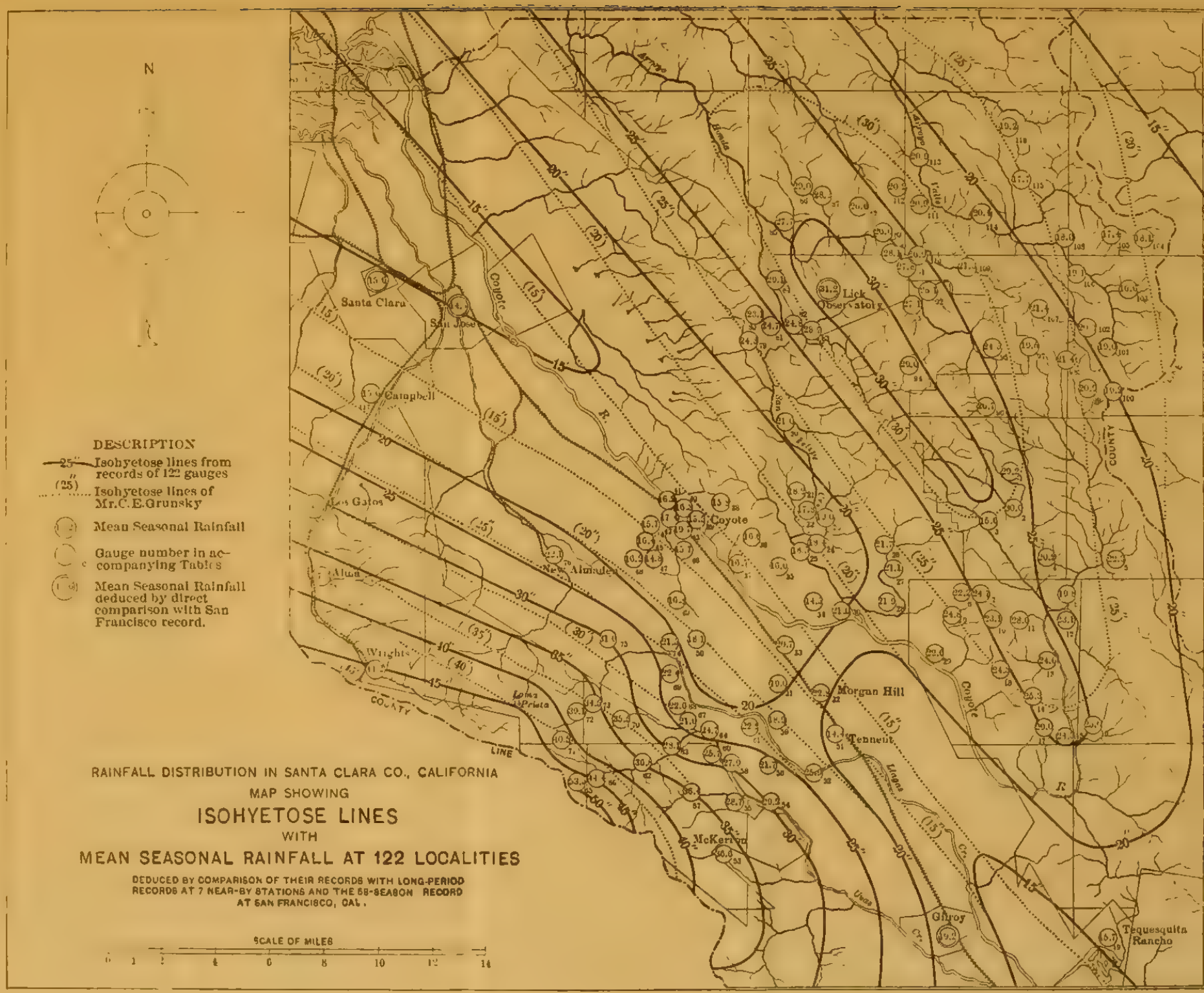


FIG. 3.

The position and extent of each group and the relation of the entire area here considered to the larger area covered by the author's map are shown by Fig. 3, which shows also the relative location of the stations, the longer-period records of which were used in deducing mean seasonal rainfalls for the stations with short-record periods. Upon Plate C are shown the location of each rain gauge, with its deduced mean seasonal rainfall, and the isohyetose lines based on these data. The author's isohyetose lines for this area are copied from his map to Plate C, thus presenting a ready means of comparison between



RAINFALL DISTRIBUTION IN SANTA CLARA CO., CALIFORNIA

MAP SHOWING

ISOHYETOSE LINES

WITH

MEAN SEASONAL RAINFALL AT 122 LOCALITIES

DEDUCED BY COMPARISON OF THEIR RECORDS WITH LONG-PERIOD RECORDS AT 7 NEAR-BY STATIONS AND THE 58-SEASON RECORD AT SAN FRANCISCO, CAL.

the author's deductions, based on data available in 1903, and the writers' deductions, based on later and more complete data now available. Messrs. Haehl and Toll.

Because most of the area covered by these records was uninhabited, it was not possible to obtain daily rainfall readings for many of the gauges. Daily records were procured, as far as possible, however, by locating gauges where they could be observed by field parties engaged in stream gauging, surveying, or construction, and by those few residents whose interest could be sufficiently aroused. In this way ten daily records were secured, in addition to those obtained from other observers, such as the Weather Bureau, the Southern Pacific Railway, etc.

Owing to the roughness of the country and the lack of wagon roads, together with the number of gauges to be observed and the distances between them, four mounted observers were constantly employed in reading gauges, with a fifth assistant for part of the time. In this way, under ordinary conditions, each gauge was visited at intervals of about one week. By special effort, during periods of unusual rainfall, it was possible to reduce the interval to 4 or 5 days for most of the gauges, while some few could not be reached for periods of several weeks during long-sustained floods, as certain streams were then impassable. Such gauges were provided with large overflow tanks, and in this way complete records were obtained.

The rain gauges were of the standard type in use by the U. S. Weather Bureau, except that they were somewhat smaller, being 3 in. in diameter. It is probable that the records are generally slightly below the true rainfall, owing to the evaporation from the gauge during the intervals between readings. This fact is best seen by comparing the records of Gauges 42 and 43 at Coyote, which were read daily, with the slightly lower records from the large number of gauges surrounding them, read weekly. The condition was suggested by records elsewhere, but the evidence is most positive in this locality, due to the many rainfall records observed in conjunction with evaporation records in this neighborhood. It has not been considered necessary, however, to attempt a correction of the records for loss by evaporation, because the error introduced is slight.

The main topographic feature of the region here considered is the Santa Clara Valley, a broad, fertile plain extending northeast and southwest midway of the area. Beginning at San Francisco Bay, this valley is about 15 miles in width; southeasterly therefrom it gradually narrows and rises, reaching its least width of about ¼ mile at Coyote, and its maximum elevation of about 350 ft. near Morgan Hill. Beyond Coyote it widens again gradually to 5 miles at Gilroy, while the elevation decreases to sea level at Monterey Bay. The influence of these changes in elevation and width upon the rainfall is clearly shown on Plate C.

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The greater precipitation shown by the records at Coyote and Morgan Hill, which the author has ignored in the construction of his isohyetose lines, is not a freak of these particular gauges, but the logical result of existing conditions, as the numerous additional records here presented clearly demonstrate. Between the Santa Clara Valley and the Pacific Ocean to the westward lies the Santa Cruz Range, the crest of which, within the area under consideration, corresponds closely to the county line shown on Plate C as part of the boundary of the area. This range, reaching an elevation of about 3 800 ft., is the first to intercept southwest storms from the ocean, and the rainfall upon it is the greatest within the area. Lying to the eastward of the Santa Clara Valley, and completing the area here considered, is the Mount Hamilton Range, which reaches a maximum elevation of more than 4 000 ft. This range consists of a series of ridges, having a northwest-southeast trend, separated by fault valleys or cañons paralleling the Santa Clara Valley, which also occupies the basin along a fault zone.

As will be noted by reference to Plate C, the rainfall upon this range is much less than upon the one to the westward. This rainfall comes for the most part with southeast winds which follow the axis of the valley from Monterey Bay. Of the moisture carried eastward from the sea the greater portion is intercepted by the Santa Cruz Range, the remainder being precipitated chiefly at the higher elevations in the Mount Hamilton Range.

The rainfall throughout this territory, as the author has pointed out, occurs chiefly during seven months of each season (November to May), more than half usually coming during December, January, and February. There is seldom any precipitation during June, July, and August. A light fall of snow, which melts rapidly, usually occurs during each season at the higher elevations in the Mount Hamilton Range, but only occasionally does snow fall in the Santa Cruz Range, and very seldom has any fallen in the foot-hills of either range, or in the valley. As a consequence of the uneven distribution of rainfall through the season, and of the comparatively steep gradients of the mountain stream beds, the greater part of each season's run-off occurs in the winter.

The two mountain ranges differ radically in their run-off characteristics, due no doubt to the difference in the forestation of the two areas. The Santa Cruz Range, except at its highest elevations, where chaparral predominates, sustains an abundant growth of timber, supported, probably, by its very heavy rainfall. In consequence, the flow of its streams responds less rapidly to heavy rainstorms, and remains well sustained through the dry season of each year. The extent of forestation in the Mount Hamilton Range is very much less, for, while the cañon bottoms and protected slopes are well covered with trees and underbrush, a considerable portion of the exposed upland sustains only

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chaparral or grass. It follows, naturally, that the greater part of the season's stream flow in this range occurs in torrents which rise rapidly after a heavy rainfall, reach their maximum flow within a few hours, and entirely recede within a few days; and also that the summer flow is almost negligible. The two ranges differ greatly, also, in the volume of run-off from like areas each year, due probably to the considerable difference in rainfall intensity in the two districts.

In deducing the mean seasonal rainfalls presented on Plate C, the local records were compared with the 58-season record at San Francisco, that being the only nearby record of more than 35 seasons' length, and the longest record in this part of the State. For many of the rain gauges, owing to their short period of record, it was necessary to make this comparison indirectly, comparing the gauge records directly with those at nearby stations, and through the medium of their longer-period records, a better basis of comparision was obtained. To determine what records to use in this intermediate capacity, a preliminary study was made of the characteristics of rainfall throughout the area. This study involved a comparison of the rainfall at various rain gauges in all parts of the area with the rainfall at the possible base stations, including the comparison of the records for each of the heavier storms as well as of the seasonal totals. The purpose, here, of course, was to determine, not what base station or combination of stations had the same quantity of rainfall as the rain gauges involved, but rather what stations were affected by the same storms and had records showing an approximately constant ratio to the gauge record. This study gave a very good basis for the selection of those base stations which would afford the most accurate means of deduction.

From this preliminary study and the writers' personal knowledge of the distribution of storms and the physical conditions influencing rainfall within the region, the rain gauges were divided into groups as follows:

Group I.—Rain gauges on the upper reaches of the Coyote River, in mountains ranging from 1 200 to 2 800 ft. in elevation, the records for which corresponded very closely in their rainfall distribution with the mean between Gilroy and Lick Observatory records.

Group II.—Rain gauges on the lower reaches of the Coyote River, on the San Felipe catchment area, and in the Santa Clara Valley, ranging in elevation from 1 900 down to 240 ft., the records of which show the same laws of distribution as the mean between the San José and Gilroy records.

Group III.—Rain gauges in the Santa Cruz Mountains between the Santa Clara Valley and the summit in the neighborhood of Loma Prieta, including the catchment areas of the Uvas and Llagas Creeks, with elevations varying from 300 to 3 800 ft. Rainfalls recorded by these gauges correspond at times to the mean of the Lick Observatory

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and Los Gatos records, at other times to the mean of the Gilroy, Santa Cruz, and Los Gatos records, while, for the period of record at the majority of the gauges of the group the mean of records at all four stations (Lick Observatory, Gilroy, Santa Cruz, and Los Gatos) gives a very reliable basis of deduction.

Group IV.—Rain gauges in the Arroyo Honda catchment area, near Mount Hamilton, at elevations from 2 200 to 3 600 ft., and on the headwaters of San Felipe Creek at 1 400 and 2 200 ft. elevation, the records of which correspond in their seasonal fluctuations with the mean of the Gilroy, Lick Observatory, and Livermore records.

Group V.—Gauges in the Arroyo Valley catchment area, at elevations varying from 1 800 to 2 600 ft., for which the best basis of deduction was found to be the mean of the Gilroy, San José, Livermore, Westley, and Lick Observatory records.

The location of the areas containing the groups of gauges just cited is shown on the map, Fig. 3, together with the positions and elevations of the various base stations selected. The conclusions as to similarity in rainfall variations between the gauges of each group and the base stations chosen in each case will be seen to find further general justification from their relative locations on this map.

Having determined what stations would be used as intermediate base stations, their 58-season mean rainfall was deduced by comparing their mean seasonal rainfall for their period of record with the mean seasonal rainfall at San Francisco for the same period, and assuming that the relation between these means also existed between the 58-season means. No single record of San Francisco rainfall could be used for this comparison, the only available record extending back to 1849 (that published by the U. S. Weather Bureau) being a composite made up of records kept successively at six different points in San Francisco. The first four of these places, having records covering the period from 1849 to 1892, had sufficiently similar rainfall conditions to warrant their being considered as one station; but the last two, with records covering the period, 1892-1907, being on the roofs of high buildings, had conditions so dissimilar to those of the first four that the use of their records in the same series with the earlier records would give an erroneous mean seasonal rainfall for San Francisco and a false basis for the deduction of the mean seasonal rainfall at other stations, the shorter-period records of which are lengthened by comparison with the San Francisco record, and means thereby determined.

It is to be regretted that in a publication of the character of Bulletin L* of the U. S. Weather Bureau no mention is made of the sources of, and the conditions affecting, the reliability of the San Francisco record there published, especially in view of the very noticeable and well-recognized difference between records taken at or near the ground.

* "Climatology of California."

and those taken at considerable elevation above the ground in the same locality. The author has called attention to the fact that the Weather Bureau record as published could not be used safely, and indicated his method of avoiding error due to its use. It is unfortunate that he was not able to publish the composite record used by him, together with those records which were the basis for it. In recent correspondence with the writers he pointed out further the break in the Weather Bureau record just referred to, and cited in full the various records used by him in forming his composite record, though he was unable to supply a copy of any of these. An attempt to collect these records developed the fact that some of them were destroyed in the Earthquake-fire of 1906. Messrs. Haehl and Toll.

The writers, therefore, have deduced a continuous, comparable San Francisco record on a different basis from that used by the author. The record sought was not one which should necessarily show the true mean rainfall for San Francisco, but one which would be strictly comparable to outside records; in other words, a record in which each season should bear the correct ratio to the mean; such ratio, and not the absolute record, being of importance as the basis for lengthening outside shorter-period records to the 58-season period, and obtaining comparable long-time means. As has been previously pointed out, the published Weather Bureau record falls short of this requirement by reason of the changed conditions governing it after 1892. However, careful rainfall records were kept in San Francisco by Mr. John Pettee, under favorable and unchanging conditions, covering 27 seasons prior to 1892 and 9 seasons thereafter; and these records form a good basis for the correction of the Weather Bureau record. Fortunately, these Pettee records are available in permanent form, having been published in Bulletin L, U. S. Weather Bureau. As will be seen by reference to Table 7, Column 4, the Pettee records bore a reasonably constant relation to the published Weather Bureau records prior to 1892, which relation for the entire period, 1865 to 1892, was 97.2 per cent. Immediately following the removal of the Weather Bureau gauge to the roof of the Mills Building, in 1892, this relation increased suddenly, and, for the 9 seasons following, averaged 134.4 per cent. The change of relation is, of course, a measure of the effect of change of elevation and location upon the Weather Bureau record, the Pettee record being consistent throughout.

To obtain a consistent 58-season record for San Francisco, accepting the Weather Bureau rainfall as published up to 1892, a record from 1892 to 1907 was deduced, therefore, which as a whole bears the same relation to the Pettee records as was borne during the 27-year period prior thereto, the rainfall varying in the individual seasons with the observed Weather Bureau records. The resulting record made use of in the deductions of means elsewhere, is given in Table 7, together with the full data used and the intermediate steps taken.

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TABLE 7.—DEDUCTION OF A CONTINUOUS, COMPARABLE, RAINFALL RECORD FOR SAN FRANCISCO, CAL.

(1)	(2)	(3)	(4)	(5)	(6)
	RECORDED RAINFALL, IN INCHES.			DEDUCED RAINFALL, IN INCHES.	
Season or period.	United States Weather Bureau.	Pettee.	Percentage relation: Pettee U. S. W. B.	Pettee.	San Francisco.
	Season.* Total.	Season.* Total.		Season.* Total.	Season.* Total.
1849-50	33.10	.....	.....	Not deduced by seasons.	33.10
50-51	7.44	.....	.....		7.44
51-52	18.44	.....	.....		18.44
52-53	35.30	.....	.....		35.30
53-54	23.84	.....	.....		23.84
54-55	23.75	.....	.....		23.75
55-56	21.68	.....	.....		21.68
56-57	19.94	.....	.....		19.94
57-58	21.97	.....	.....		21.97
58-59	22.03	.....	.....		22.03
59-60	22.46	.....	.....		22.46
60-61	19.51	.....	.....		19.51
61-62	49.27	.....	.....		49.27
62-63	13.74	.....	.....		13.74
63-64	10.29	.....	.....		10.29
64-65	24.52	.....	.....		24.52
1849-65	367.28		(assumed) 97.2	357.07	Same as U. S. W. B. record. 367.28
1865-66	22.93	23.57	102.7	Same as Pettee recorded rainfall.	22.93
66-67	34.92	35.94	102.9		34.92
67-68	38.84	40.62	104.5		38.84
68-69	21.35	20.66	96.7		21.35
69-70	19.31	20.12	104.2		19.31
70-71	14.13	13.12	92.9		14.13
71-72	30.77	28.89	93.9		30.77
72-73	15.74	19.59	124.4		15.74
73-74	24.64	24.50	99.4		24.64
74-75	20.56	18.15	88.3		20.56
75-76	31.21	28.31	90.7		31.21
76-77	11.04	9.98	90.4		11.04
77-78	35.17	32.80	93.2		35.17
78-79	24.46	22.14	90.5		24.46
79-80	26.63	23.61	88.6		26.63
80-81	29.86	27.24	91.2		29.86
81-82	16.14	15.83	98.0		16.14
82-83	20.12	19.59	97.3		20.12
83-84	32.42	29.18	90.0		32.42
84-85	18.12	17.07	94.2		18.12
85-86	31.50	28.75	91.2		31.50
86-87	18.82	16.68	88.6		18.82
87-88	16.75	16.95	101.1		16.75
88-89	23.85	24.25	101.6		23.85
89-90	45.86	46.43	101.2		45.86
90-91	17.68	18.74	106.5		17.68
91-92	18.41	20.13	108.9		18.41
1865-92	661.23	642.84	97.2	642.84	Same as U. S. W. B. recorded rainfall. 661.23

* The season here used is from September to August, inclusive.

TABLE 7.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)
	Recorded rainfall, in inches.			Deduced rainfall, in inches.	
Season or period.	United States Weather Bureau.	Pettee.	Percentage relation: Pettee / U. S. W. B.	Pettee.	San Francisco.
	Season.* Total.	Season.* Total.		Season.* Total.	Season.* Total.
1892–93	21.77	27.15	124.7	Same as Pettee recorded rainfall.	30.09
93–94	18.45	24.44	132.5		25.51
94–95	25.71	34.94	135.8		35.55
95–96	21.37	29.45	137.8		29.54
96–97	23.30	30.91	132.6		32.21
97–98	9.38	13.83	147.4		12.97
98–99	16.87	23.96	142.0		23.32
99–00	18.47	25.40	137.5		25.53
00–01	21.17	27.11	128.1		29.25
1892–1901	176.49	237.19	134.4	237.19	= U. S. W. B. × 134.4% ÷ 97.2%
1901–02	18.98	.....	.....	25.50	26.24
02–03	18.28	.....	.....	24.58	25.27
03–04	20.67	.....	.....	27.78	28.58
04–05	23.37	.....	.....	31.41	32.31
05–06	20.61	.....	.....	27.71	28.49
06–07	25.98	.....	.....	34.90	35.91
1901–07	127.89	.....	(assumed) 134.4	171.88	
1892–1907	304.38			409.07	420.77
1849–1907	1 332.89	.....	.....	1 408.98	1 449.28
58-Season Mean	22.98	.....	.....	24.29	24.99

* The season here used is from September to August, inclusive.

The rainfalls shown in Column 2 of Table 7 are taken from an advance sheet published by the San Francisco office of the U. S. Weather Bureau in 1908, as the San Francisco rainfall. This record is a composite, stated by Professor A. G. McAdie, under whose direction it was published, to be the record kept by Mr. Thomas Tennent, maker of nautical instruments, from 1849 to 1871, and the U. S. Signal Service and U. S. Weather Bureau records thereafter. These records were obtained under similar conditions from 1849 to 1892. The records from 1892 to 1907, however, form a series not comparable with the records for the earlier period.

The rainfalls shown in Column 3 are taken from Bulletin L, U. S. Weather Bureau, and represent a record of rainfall in San Francisco, carefully kept by Mr. John Pettee at a single guage, near the ground, under unchanging conditions.

The percentages in Column 4, showing the relations between seasonal rainfalls in the continuous and the discontinuous record, clearly

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illustrate the break of 1892 in the Weather Bureau record, and indicate means of deducing a continuous record for the entire period from 1849 to 1907.

Column 5 shows the Pettee record lengthened to cover the 58 seasons by the use of two assumptions: (1) That the Pettee record would have shown the same relation to the U. S. Weather Bureau record from 1849 to 1865 which it did show for the period, 1865-1892; (2) That for the seasons, 1901-02 to 1906-07, the Pettee record bore to the U. S. Weather Bureau record the relation shown for the period 1892-1901. (See Column 4.)

Column 6 shows the deduced San Francisco record. This is the same as the U. S. Weather Bureau record in Column 2 from 1849 to 1892. The rainfall from 1892 to 1907 is deduced on the assumption that the deduced Pettee rainfall in Column 5 bears the same relation to the deduced San Francisco rainfall for the period 1892-1907 as it bears for the period, 1849-1892 (97.2%). Since this deduced Pettee rainfall is 134.4% of the U. S. Weather Bureau recorded rainfall for the later period, it follows that the relation between the deduced continuous San Francisco rainfall and the recorded U. S. Weather Bureau rainfall for the period, 1892-1907, is $134.4 \div 97.2 = 138.3$ per cent. This percentage relation is applied to each season's record from 1892-93 to 1906-07, given in Column 2, to obtain that given in Column 6.

As understood from Mr. Grunsky's paper, the San Francisco record used by him was an average of all available records for each season, the number of records varying from season to season, in some cases being but one and in others as many as five. It is believed that, for the end sought, the method followed by the writers is more logical, though possibly less representative of the mean rainfall in San Francisco.

The mean seasonal rainfall for each locality, as given on Plate C, was deduced, in the same manner as for base stations, by comparing the total recorded rainfall at the gauge with the total for the same period shown by the mean of the base stations used. It was thought best not to determine and compare short-period means, but, instead, to make the comparisons by totals in this case, since two sets of fractional seasons were used, namely: (1) January to August, 1903, inclusive, for 22 gauges having readings which were begun in December, 1902, and (2) September, 1907, to March, 1908, inclusive, for 5 gauges having records which were available up to the latter date. The stations having long-period records which were here used as the basis of deduction are given in Table 8, together with the period and length of each record and the steps taken in deducing the 58-season mean from the record at San Francisco. In Tables 9 to 13 are given the records for gauges in Groups I to V, respectively. These tables contain also the elevations of each gauge (obtained in most cases by repeated aneroid readings and hence fairly reliable), the number of seasons of record, and the steps taken in deducing each long-period mean, as has just been explained.

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TABLE 8.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

BASE STATIONS.

Using San Francisco Record as Basis.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Station.	Period of record.	Number of seasons.	MEAN SEASONAL RAINFALL FOR PERIOD OF STATION RECORD, IN INCHES.		Percentage relation: Col. 4 / Col. 5	Deduced 58-season mean at station, in inches.
			At station.	At San Francisco.		
San Francisco....	'49-50 to '06-07	58	24.99	24.99	100.0%	24.99
Santa Clara....	'81-82 " '84-85, '87-88 " '06-07	24	16.33	26.25	62.2	15.55
San José.....	'74-75 " '06-07	33	15.12	26.10	57.9	14.47
Los Gatos.....	'85-86 " '06-07	22	34.09	27.06	125.9	31.46
Santa Cruz.....	'78-79 " '06-07	29	27.28	26.31	103.6	25.90
Gilroy.....	'74-75 " '06-07	33	20.10	26.10	77.0	19.24
Lick Observatory	'81-82 " '06-07	26	32.77	26.23	124.8	31.20
Livermore.....	'71-72 " '00-01, '02-03 " '06-07	35	15.76	25.89	60.8	15.20
Westley.....	'89-90 " '06-07	18	10.74	27.93	38.4	9.60

TABLE 9.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP I; GAUGES 1-19.

Using Mean of Gilroy and Lick Observatory Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with approximate elevation).	Period of record.		TOTAL RECORDED RAINFALL FOR PERIOD OF GAUGE RECORD, IN INCHES.		Relation: Col. 3 / Col. 4	58-SEASON MEAN, IN INCHES.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
1 (1870)	Jan. '03-Aug. '05	2½ Seasons.	80.46	69.44	115.8%	25.22	29.2
2 (1800)	" "	2½ "	82.69	.....	119.0	.....	30.0
3 (2730)	" "	2½ "	43.02	.....	61.9	.....	15.6
4	" "	2½ "	55.50	.....	79.9	.....	20.2
5 (2440)	" "	2½ "	61.51	.....	88.6	.....	22.3
6 (2500)	" "	2½ "	61.23	.....	88.2	.....	22.2
7 (2800)	" "	2½ "	67.93	.....	97.8	.....	24.7
8 (1350)	" "	2½ "	54.40	.....	78.3	.....	19.8
9 (1720)	" "	2½ "	68.26	.....	98.3	.....	24.8
10 (2230)	" "	2½ "	63.60	69.44	91.6	.....	23.1
11 (1490)	Sep. '03-Aug. '06	3 "	97.26	85.84	113.3	.....	28.6
12 (1240)	Sep. '03-Aug. '05	2 "	47.59	51.92	91.6	.....	23.1
13 (1220)	Jan. '03- "	2½ "	67.69	69.44	97.5	.....	24.6
14 (2720)	" "	2½ "	69.71	.....	100.4	.....	25.3
15 (1210)	" "	2½ "	66.82	.....	96.2	.....	24.3
16 (1200)	" "	2½ "	68.82	69.44	99.1	.....	25.0
17	Sep. '03- "	2 "	59.58	51.92	114.8	.....	29.0
18 (2780)	" "	2 "	50.01	51.92	96.3	.....	24.3
19	Sep. '99—Aug. '06	7 "	113.48	182.02	62.3	25.22	15.7

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TABLE 10.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP II; GAUGES 20-51.

Using Mean of Gilroy and San José Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with approximate elevation).	Period of record.		TOTAL RECORDED RAINFALL FOR PERIOD OF GAUGE RECORD, IN INCHES.		Relation: Col. 3 / Col. 4	58-SEASON MEAN, IN INCHES.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
20 (1050)	Jan. '03–Aug. '05	2½ Seasons.	57.63	46.36	124.4%	16.86	21.0
21	Sep. '03– "	2 "	37.67	34.97	107.6	.....	18.2
22	" "	2 "	37.01	34.97	105.9	.....	17.8
23 (750)	Jan. '03– "	2½ "	52.16	46.36	112.5	.....	19.0
24 (650)	" "	2½ "	50.44	46.36	108.9	.....	18.4
25 (640)	Sep. '03– "	2 "	38.36	34.97	109.6	.....	18.5
26 (1250)	Jan. '03– "	2½ "	59.57	46.36	128.4	.....	21.7
27 (1300)	" "	2½ "	57.85	46.36	124.8	.....	21.1
28 (1560)	" –Aug. '06	3½ "	89.26	68.62	130.0	.....	21.9
29 (1870)	Sep. '03–Aug. '05	2 "	46.78	34.97	133.8	.....	22.6
30 (440)	" –Mar. '08	5 "	118.35	95.11	124.4	.....	21.0
31	" –Aug. '05	2 "	39.43	34.97	112.8	.....	19.0
32 (350)	Sep. '99–Aug. '04	5 "	135.67	100.36	135.2	.....	22.8
33 (345)	Jan. '03–Mar. '08	5½ "	130.51	106.50	122.5	.....	20.7
34	Sep. '03–Aug. '05	2 "	30.04	34.97	85.9	.....	14.5
35 (1175)	" "	2 "	34.45	.....	98.5	.....	16.6
36	" "	2 "	34.44	34.97	98.5	.....	16.6
37 (310)	" –Aug. '07	4 "	81.47	83.09	98.0	.....	16.5
38	" –Aug. '05	2 "	32.79	34.97	93.7	.....	15.8
39 (248)	" –Aug. '07	4 "	76.56	83.09	92.2	.....	15.5
40 (246)	Jan. '03–Aug. '05	2½ "	44.94	46.36	96.9	.....	16.3
41 (242)	Sep. '03– "	2 "	33.74	34.97	96.4	.....	16.2
42 (248)	" –Mar. '08	5 "	98.37	95.11	103.4	.....	17.4
43 (255)	Sep. '99–Aug. '04	5 "	117.32	100.36	116.9	.....	19.7
44 (310)	Sep. '03–Aug. '07	4 "	77.50	83.09	93.3	.....	15.7
45 (250)	" "	4 "	80.61	83.09	97.0	.....	16.4
46 (300)	" –Aug. '06	3 "	53.37	57.24	93.3	.....	15.7
47	Sep. '04–Aug '05	1 "	18.01	20.48	87.9	.....	14.8
48	" "	1 "	19.64	.....	96.0	.....	16.2
49	" "	1 "	20.35	.....	99.4	.....	16.8
50	" "	1 "	21.96	20.48	107.2	.....	18.1
51 (325)	Sep. '78–Aug. '94	16 "	239.04	280.33	85.3	16.86	14.4
117 *(195)	Sep. '97–Aug. '04	7 "	97.84	*142.98	68.0	*22.96	15.6

*Also Gauge 117, using mean of San José and Los Gatos records as basis.

Having located all the available rainfall record stations on Plate C, the isohyetose lines were drawn in the manner described by the author, giving due weight to each record in accordance with its length and the influence of its known location upon its value as a representative record. It may be noted that in very few instances are the local records at great variance with the isohyetose lines as drawn. The most notable instance of such disagreement is Gauge No. 3 on the Upper Coyote, surrounded by gauges showing means varying from 20 to 30

in., and yet giving by its record only 15.6 in. This difference is not due to any misreading of the gauge, for the record consistently bears about the same ratio to records of neighboring gauges for every storm, and no reason is apparent for its disagreement. Other examples of disagreement are Gauge No. 34 and Gauge No. 51, the latter located at Tennent. The Tennent gauge has been given considerable weight on account of its longer record, though it is seemingly much too low, and nothing is known to the writers concerning the care with which it was located or observed. Messrs. Haehl and Toll.

TABLE 11.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP III; GAUGES 52-78.

Using Mean of Gilroy, Lick Observatory, Santa Cruz, and Los Gatos Records as Basis.

(1)		(2)		(3)	(4)	(5)	(6)	(7)
Gauge No.	(with approximate elevation).	Period of record.		TOTAL RECORDED RAINFALL FOR PERIOD OF GAUGE RECORD, IN INCHES.		Relation: Col. 3 / Col. 4	58-SEASON MEAN, IN INCHES.	
				At gauge.	Mean of base stations		For base stations.	For gauge (deduced).
52	(360)	Sep. '06-Mar. '08	2 Seasons	55.25	57.45	96.2%	26.95	25.9
53		Sep. '92-Aug. '02	10 "	377.40	277.64	135.9	.....	36.6
54	(535)	Sep. '03-Aug. '05 / Sep. '06-Mar. '08	4 "	125.45	115.77	108.4	.....	29.2
55	(360)	Sep. '03-Aug. '05	2 "	62.18	58.32	106.6	.....	28.7
56	(640)	" "	2 "	47.05	.....	80.7	.....	21.7
57	(470)	" "	2 "	78.67	.....	135.0	.....	36.4
58	(460)	" "	2 "	60.40	.....	103.6	.....	27.9
59	(580)	" "	2 "	40.91	.....	70.1	.....	18.9
60	(340)	" "	2 "	55.72	.....	95.5	.....	25.7
61	(690)	" "	2 "	47.92	.....	82.2	.....	22.2
62	(845)	" "	2 "	79.55	.....	136.5	.....	36.8
63	(880)	" "	2 "	60.76	.....	104.2	.....	28.1
64	(620)	" "	2 "	53.55	.....	91.8	.....	24.7
65	(1 900)	" "	2 "	115.75	.....	198.5	.....	53.5
66	(965)	" "	2 "	95.50	.....	163.8	.....	44.2
67	(660)	" "	2 "	45.51	.....	78.0	.....	21.0
68	(732)	" "	2 "	48.90	.....	83.9	.....	22.6
69	(730)	" "	2 "	48.51	.....	83.2	.....	22.4
70	(930)	" "	2 "	76.73	.....	131.6	.....	35.5
71		" "	2 "	87.54	.....	150.2	.....	40.5
72	(1 190)	" "	2 "	84.63	58.32	145.2	.....	39.1
73		" -Aug. '04	1 "	35.41	27.70	127.8	.....	34.5
74	(745)	" -Aug. '05	2 "	46.57	58.32	79.8	.....	21.5
75	(910)	" "	2 "	67.01	58.32	114.9	.....	31.0
76		Sep. '87-Aug. '04	17 "	388.94	474.26	81.9	.....	22.1
77		Sep. '99-Aug. '03	4 "	176.60	107.71	163.8	.....	44.2
78		Sep. '00-Aug. '04	4 "	152.66	111.32	137.1	26.95	37.0

A study of the isohyetose lines on Plate C will show how greatly the topography affects the distribution of rainfall. The effect of the gradual rise in the valley floor toward Morgan Hill has been referred

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to, and attention is called to the variations in the lines of equal rainfall caused by the ridge extending northeasterly from Loma Prieta, the variation being more accentuated by the influence of the Uvas Creek Cañon, paralleling the ridge to the south. It may be noted, further, that the series of fault cañons in the Mount Hamilton Range have decidedly influenced the distribution of rainfall in that region. These effects of the topography are not only shown by the records here presented, but are known to the inhabitants of the region, and have been recognized by the writers from general observations extending through several years.

TABLE 12.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP IV; GAUGES 79-97.

Using Mean of Gilroy, Lick Observatory, and Livermore Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record, in Inches.		Relation: Col. 3 / Col. 4.	58-Season Mean, in Inches.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
79 (1400)	Sep. '03–Aug. '05	2 Seasons.	49.28	44.32	111.1%	21.88	24.3
80 (2200)	" –Aug. '04 / Sep. '05–Aug. '06	2 "	53.84	50.97	105.7	.....	23.1
81 (2040)	" "	1 "	32.75	29.06	112.7	.....	24.7
82 (2350)	Sep. '03–Aug. '04	1 "	24.80	21.91	113.2	.....	24.8
83 (2350)	" –Aug. '05	2 "	58.08	44.32	131.1	.....	28.7
84 (2460)	" "	2 "	58.93	44.32	133.0	.....	29.1
85 (2350)	" –Aug. '07	4 "	133.24	105.20	126.7	.....	27.7
86 (2450)	" "	4 "	139.59	105.20	132.7	.....	29.0
87 (2900)	" –Aug. '05	2 "	57.08	44.32	128.8	.....	28.2
88 (2950)	" "	2 "	53.81	.....	121.5	.....	26.6
89 (2250)	" "	2 "	52.63	.....	118.8	.....	26.0
90 (2275)	" "	2 "	56.81	.....	128.3	.....	28.1
91 (2250)	" "	2 "	56.35	.....	127.2	.....	27.8
92 (2350)	" "	2 "	52.74	.....	119.1	.....	26.1
93 (2300)	" "	2 "	54.88	44.32	123.9	.....	27.1
94 (3625)	" –Aug. '04	1 "	29.00	21.91	132.4	.....	29.0
95 (2330)	" "	1 "	24.57	21.91	112.1	.....	24.5
96 (2650)	" "	1 "	26.75	21.91	122.1	.....	26.7
97 (2550)	" –Aug. '05	2 "	39.54	44.32	89.2	21.88	19.5

In comparing the isohyetose lines drawn by the writers with those given by the author (also shown on Plate C), it will be noted that three general differences exist:

1.—The writers' 15-in. and 20-in. lines do not cross the valley summit between San José and Gilroy, a gap of more than

20 miles intervening between the two 15-in. lines in the valley. Messrs. Haehl and Toll.

2.—The writers show a mean seasonal rainfall of more than 50 in. at the crest of the Santa Cruz Range, as compared with the author's maximum of 45 in.

3.—The rainfall indicated by the local records falls off in quantity much more rapidly to the northeast of Mount Hamilton (Lick Observatory) than shown by the author's lines.

TABLE 13.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP V; GAUGES 98–116.

Using Mean of Gilroy, San José, Livermore, Westley, and Lick Observatory Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record.		Relation: Col. 3, Col. 4.	58-Season Mean, in Inches.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
98 (2130)	Sept. '03–Aug. '05	2 Seasons	43.06	36.15	119.2%	17.94	21.4
99 (2150)	" "	2 "	42.05	.....	116.3	.....	20.9
100 (2550)	" "	2 "	38.67	.....	107.0	.....	19.2
101 (2160)	" "	2 "	38.29	.....	105.9	.....	19.0
102 (2110)	" "	2 "	40.79	.....	112.9	.....	20.3
103 (2250)	" "	2 "	33.45	.....	92.6	.....	16.6
104 (2450)	" "	2 "	36.51	.....	101.0	.....	18.1
105 (2290)	" "	2 "	35.05	.....	96.9	.....	17.4
106 (2140)	" "	2 "	38.46	.....	106.4	.....	19.1
107 (2250)	" "	2 "	43.18	.....	119.5	.....	21.4
108 (2120)	" "	2 "	36.33	.....	100.5	.....	18.0
109 (2030)	" "	2 "	42.91	.....	118.8	.....	21.3
110 (2450)	" "	2 "	54.21	.....	150.0	.....	26.9
111 (2100)	" "	2 "	42.06	.....	116.4	.....	20.9
112 (2500)	" "	2 "	40.70	36.15	112.6	.....	20.2
113 (1800)	" –Aug. '04	1 "	19.59	16.83	116.4	.....	20.9
114 (2000)	" –Aug. '05	2 "	41.08	36.15	113.7	.....	20.4
115 (2260)	" "	2 "	35.58	.....	98.4	.....	17.7
116 (2460)	" "	2 "	38.73	36.15	107.2	17.94	19.2

The minor disagreements between many of the isohyetose lines of the writers and of the author are due, of course, to the greater number of data available to the former, and it is worthy of notice that the addition of more than 100 records within this area has made so few changes, and has so nearly confirmed the lines of the author. The general effect of the changes made is to show a greater mean seasonal rainfall for the Santa Clara Valley and the Santa Cruz Mountains to

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the westward, and a less rainfall for most of the Mount Hamilton Range to the eastward.

The correctness of the location of the 15-in. lines just east of San José, and in the extreme northeasterly portion of the area shown on Plate C, is still comparatively uncertain from lack of adequate data fixing these lines. It is to be regretted that no rainfall records (so far as known to the writers, at least) have been kept in these localities.

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PAPERS AND DISCUSSIONS

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THE MAINTENANCE OF MACADAM AND OTHER ROADS.

An Informal Discussion at the Annual Convention, June 25th, 1908.*

By Messrs. Arthur H. Blanchard, John R. Rablin, Louis L. Tribus, James Owen, Nelson P. Lewis, G. N. Houston, and D. C. Wedgeworth.

Mr. Blanchard.

Arthur H. Blanchard, Assoc. M. Am. Soc. C. E.—Although the subject of this discussion covers the maintenance of various types of surfaces built in the country districts, and hence includes earth, gravel, and macadam roads, this introductory discussion will be devoted exclusively to the surface construction and the maintenance of macadam roads subjected to modern traffic.

Five years ago the highway engineer had to contend with the action of the elements, poor foundations, narrow tires, heavy loads, and road material deficient either in resistance to abrasion, in hardness, toughness, or in cementing properties. Since then the nature of the traffic to which macadam roads have been subjected has changed materially. The advent of the swiftly-moving motor car has necessitated a readjustment of methods, both of construction and maintenance.

In order to facilitate an intelligent discussion of the subject of the maintenance of macadam roads, from the standpoints of economics and efficiency, it is advisable to consider a classification of suburban and country roads, especially in those States in which a large percentage of the main highways is used by motorists. Otherwise, that important and fundamental problem, the adaptation of macadam roads to modern traffic, will be lost sight of in the discussion of

* Further discussion, had at the Convention, will be published subsequently and additional written discussions are invited.

Mr. Blanchard. various methods experimented with for the allaying of dust and the preservation of highways.

It is practicable to recognize three classes of highways existing outside of the city limits, the classification being based on the amount and character of the traffic to which the roads are subjected. Naturally, these classes merge into and overlap each other in many cases, but the classification will suffice as a basis for comparison. The first class includes interstate trunk lines, interurban trunk lines, and popular routes of travel. The second class includes intrastate trunk lines passing through towns, the highways connecting towns situated within a few miles of each other, and secondary streets in towns. The third class includes feeders leading to towns, and highways of the first and second classes from sparsely-populated parts of the country districts, highways connecting towns which are many miles apart, cross-roads, and transverse feeders in towns.

Highways of the first class will undoubtedly be subjected to the heaviest motor-car traffic, although not necessarily to the heaviest commercial traffic. Highways of the second class will probably carry heavy commercial traffic and a limited amount of motor-car traffic. The third class of highways will usually be subjected, in amount, at least, to the ordinary country highway traffic. The maintenance of highways of the third class, which are constructed either of earth, gravel, or broken stone, will not be considered, as it is intended to devote this discussion to the maintenance of those highways of the first and second classes which, being subjected to a more or less heavy motor-car traffic and an augmented commercial traffic, demand an entirely different method of maintenance, and in many cases a modified construction of the broken stone surface. As many macadam streets and park drives within the limits of municipalities are subjected to the same kind and amount of traffic as highways of the first and second classes, the methods of maintenance will be similar. The amount of traffic and the speed attained by motor cars are two of the most important factors to be considered in the discussion of methods for maintaining highways of the first and second classes.

The problem before the highway engineer is to determine what type of road surface for highways of the first and second classes will be most economical from the standpoints of first cost, maintenance, and cost of renewal, and best adapted to the traffic to which it is subjected. In considering methods of construction and maintenance, it is essential to have definitely in mind what is expected to be accomplished by a given mode of procedure. Unfortunately, this point of view has been lost sight of by many who are responsible for the maintenance of urban, suburban, and even country highways. The distinction between the use of various materials as palliatives and dust layers, and the use of the same materials as component parts of more permanent construction is exceedingly important.

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Considering the modern methods of maintenance used in the United States by proceeding from the simple to the complex, the common remedy for a road from which the binder has been removed is to supply the deficiency by spreading an efficient binding material such as sand, gravel, or broken-stone screenings over the middle of the road and allowing traffic to work the binder into the interstices of the top course. This method is usually effective, but is not efficacious for highways of the first class subjected to excessive motor-car traffic. Strenuous objection is naturally made to this mode of procedure by the residents adjacent to the highway, as the dust nuisance is considerably increased.

In order of simplicity, the methods of retaining the dust and thus aiding in the preservation of the surface should next be mentioned. Almost everywhere, the following methods have been used primarily to allay the dust, although in some cases they have been used for strictly maintenance purposes. Among the methods used in the United States to alleviate the dust nuisance may be mentioned, sprinkling the surface with fresh water, salt water, a solution of calcium chloride, oils with a paraffin base, oils with an asphaltic base, oil of tar, oil emulsion, Westrumite, Dustoline, Asphaltoline, Tarracolio, and deliquescent salts. The methods of painting or spraying the surface with various coal-tar, asphalt, and cement mixtures have been used, both for the preservation of the surface and the elimination of dust. The details of the methods of using the materials mentioned will not be considered in this introductory discussion, except in relation to experimental work in Rhode Island. It is hoped that the practice in other States will be presented by highway engineers who are conversant with maintenance work in the districts which they represent.

Thus far, the methods of constructing more permanent macadam roads have been limited to the use of coal-tar or its preparations, liquid and rock asphalt, and cement as binding materials. Bituminous macadam pavements, of which the modern bitulithic is an example, were first laid in America about 1840. The cost of these pavements prohibits their use except on the streets of cities and important suburbs. Two fundamental methods have been used in the construction of bituminous macadam applicable to country roads. First, to incorporate the binding material with the broken stone by mixing by hand or machinery, and second, to form the macadam surface, loose or rolled as the case may be, and distribute the binding material by dippers or machines over the broken stone, allowing the fluid to penetrate the upper course.

The reconstruction of macadam roads has been accomplished by loosening the surface with the picks of a road roller, or by the use of a scarifier, and afterward impregnating the broken stone with a

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coal-tar preparation, or with liquid asphalt, or a heavy asphaltic oil. In case re-surfacing is necessary, the same method is followed, except that practice differs as to whether or not the new material should be shaped by light rolling before the application of the bituminous binder.

The engineers of France and England have been studying the problem of the adaptation of macadam roads to modern traffic for the past five years, and hence have advanced along some lines of research farther than American engineers. Although many patented solutions have been tried in France and England, most engineers favor the use of a bituminous binder in some form as an integral part of new construction and as a palliative on old macadam roads. The methods, however, vary to a considerable degree. In both countries the introduction of various tar-spreading machines has materially increased the efficiency of the tar-painting method and reduced the cost of applying the tar. The Laissailly machine, used in France, is typical of machines used in both countries. It consists of a mounted tank containing the tar, which is heated by a system of piping through which steam passes. From a tank on the rear of the vehicle, which serves as a reservoir, the tar flows by gravitation out of a series of holes, and is distributed by automatic brushes attached to the body of the vehicle. Devices for applying the tar under pressure form integral parts of other machines, with which it is claimed that a uniform penetration of from 1 to 3 in. can be secured.

In addition to the mixing and penetration methods of constructing tar-macadam roads, English engineers are using the Gladwell system or modifications of this method. This system consists in spreading on the No. 1 course, after rolling, a mixture of tar and chips to a depth of about ¾ in. On this is deposited from 2 to 3 in. of No. 2 stone, which is continually rolled with a light-weight roller until the tar mixture has been drawn up into the interstices of the No. 2 course. The surface is then sealed by painting with tar, after which a thin coat of fine screenings is spread and rolled. In the construction of all kinds of tar-macadam, careful attention is paid to the quality of the material used, particularly with reference to the tar-absorbing power of the material. The experience of English engineers is that iron slag, the basis of Tarmac, stands first in this respect, limestone next, and granite last.

The primary object of the experimental work undertaken in Rhode Island has been the determination of the most economical and efficient method of securing the preservation of the macadam surface.

In the fall of 1906 a section of tar-macadam, 350 ft. in length, was built in Charlestown, the location being on a curve of the interstate trunk line connecting New York, Narragansett Pier, Providence, and Boston. With the exception of the addition of the tar, the method

of construction was similar to that used in building an ordinary macadam road. After the sub-grade had been thoroughly rolled, the No. 1 broken stone, varying in size from 1¼ to 2½ in. in longest dimension, was spread to a depth of 6 in. and rolled to 4 in. Tar, which had been heated to the boiling point in an ordinary tar kettle, was then sprinkled on the rolled surface by using dippers. The No. 2 stone, varying in size from ½ in. to 1¼ in. in longest dimension, was next deposited on dumping boards and thoroughly mixed with hot tar by using rakes and shovels until every stone was completely coated. This mixture was applied on the No. 1 course to a depth of 3 in. and, after the tar had set, was rolled to 2 in. A thin coat of dust, which would pass through a ½-in. mesh was then spread on the surface and forced by rolling into the No. 2 course to fill up the voids and provide a smooth surface. The quantity of tar used was 1.25 gal. per sq. yd. As this stretch of tar-macadam proved efficacious, it was deemed advisable, in 1907, to construct an experimental mile, this section being on the same interstate trunk line as the curve just mentioned. Although it will require from 5 to 10 years to determine the economical status and the efficiency of the tar-macadam road constructed by the method outlined, the perfect state of the surface of the 1906 and 1907 sections in the spring of 1908 influenced the State engineers to advocate the adoption of this method of construction on highways of the first class subjected to excessive motor-car traffic. Mr. Blanchard.

The sections thus far contracted for in 1908 include: First, two sections aggregating 11 300 ft., 14 ft. in width, on the interstate trunk line between Saunderstown and Narragansett Pier, which is subjected to all the motor-car traffic between the Pier and Newport, Providence, and Boston, and the heavy touring-car traffic between New York City and Newport; and second, 11 870 ft., averaging 23 ft. in width, on an interstate trunk line in East Providence, which is subjected to a heavy motor-car traffic and also an abnormal commercial traffic. On one of the sections in the district of Narragansett the State engineers experimented with various bituminous mixtures used in different methods of construction, the object being to determine the most efficient binder and the most economical method of construction which would be adaptable to modern traffic. The original plan was to build, by each method, a 300-ft section, 14 ft. in width, but the impracticability of certain methods with the tools at hand necessitated a modification of this plan. The experimental work was started after 1 300 ft. of tar-macadam had been constructed in accordance with the method used in 1906 and 1907. Table 1 is a descriptive statement of the various sections.

In the following statements, covering details and cost data of the various sections, it is hoped that sufficient information will be given

Mr. Blanchard. to enable engineers to use the results of these tests intelligently. It has been characteristic of many descriptive articles dealing with maintenance work that the data necessary to render the results acquired valuable to others have been lacking. Although the ideal is impossible of attainment in many cases, it would be advisable if the following points, for example, could be covered in a description of methods of construction of tar-macadam: Class of highway to which the road belongs, and nature of traffic; length and width of road; nature of sub-grade; location of the road relative to the nearest railroad station and source of material; maximum and mean grade of road; kind and size of broken stone composing the surface; months during which road was constructed; maximum, minimum, and mean atmospheric temperature; details of construction; source and analysis of binding material; quantity of binder per square yard; average rate of progress per day, in square yards; specifications covering heating and application of the binder, climatic conditions, etc.; cost per square yard, over and above the cost of ordinary macadam, covering labor, material, accessories, etc.

TABLE 1.

Number of section.	Length, in feet.	Material.	Method, etc.
1	300	Tar-macadam.	Mixing method, with tar omitted from surface of No. 1 course.
2	300	Tar-macadam.	Penetration method.
3	100	Tar-macadam.	Gladwell system.
4	300	Tar-macadam.	Mixing method.
5	130	Tar-pitch-macadam.	Mixing method, with chips added to No. 2 mixture.
6	70	Tar-pitch-macadam.	Mixing method.
7	300	Tar-macadam.	Mixing method.
8	300	Tar-asphalt-macadam.	Mixing method.
9	290	Tar-asphalt-macadam.	Mixing method, with bituminous mixture omitted from No. 1 course.
10	75	Asphalt-macadam.	Mixing method.
11	135	Tar-asphalt-macadam.	Mixing method, without bituminous mixture on surface of No. 1 course.
12	410	Tar-asphalt-macadam.	Mixing method.
13	190	Tarvia-macadam.	Mixing method.
14	172	Tarvia-macadam.	Mixing method, with Tarvia omitted from No. 1 course.

The roadbed selected for the experimental work was on a long tangent, the sub-grade being uniformly built on a 1-ft. fill, and the grade only varying from a minimum of 0.08% to a maximum of 0.76 per cent. The broken stone used was of the same character throughout, consisting of local stone composed of a fine-grained granite and hornblende schist of the same size as that used on the 1906

and 1907 sections. The experimental work was all done in the month of May. The temperature while the work was in progress varied from 60 to 90°, the average being 70 degrees. The tar used was produced at the Providence Gas Works, and cost, f. o. b. Providence, $2.75 per bbl. The freight from Providence to South Ferry, a distance of 26 miles, distributed over three lines, was $0.62 per bbl. The cost of the haul from the South Ferry station to the road, averaging 2 000 ft., was $0.13 per bbl. The barrels cost $0.75 each, which, minus the haul and the freight to Providence, would be refunded by the gas company upon return of the barrels. The net rebate was $0.56 per bbl. The total cost of the tar, therefore, was $3.70 per bbl. Mr. Blanchard.

The coal-tar produced at the Providence Gas-Works is characteristic of gas-house coal-tars produced elsewhere, in that the percentage of water, ammoniacal liquor, light volatile oils and heavy oils is a variable quantity. In order to determine various facts, including the range of variation, samples, taken directly from the tanks at the gas-works at various times, were analyzed. Table 2 shows the important results of the analyses. The percentage of volatile matter at 120° after heating from 6 to 8 hours represents the percentage which might be lost from the tarred surface of a road during summer seasons. The results in the column headed "Solid matter, in grains per gallon," were obtained by digesting a portion of the sample with an equal volume of water. The object of this analysis was to determine the quantity of matter which might be washed from the surface of a road by a heavy rain. The average specific gravity of the tar was about 1.26.

TABLE 2.—Coal-Tar Analyses.

Date.	Percentage of volatile matter at 120 degrees.	Percentage of volatile matter at 260 degrees.	Solid matter, in grains per gallon.
Nov. 20, 1907	10.6	30.7	46.7
Nov. 27, 1907	6.8	27.9	102.2
Dec. 18, 1907	11.1	34.9	105.1
Dec. 24, 1907	7.1	21.3	20.4
Jan. 14, 1908	16.3	32.3	52.5
Jan. 21, 1908	7.2	19.4	5.8
Feb. 19, 1908	5.8	16.2	2.9
Feb. 26, 1908	4.7	15.3	8.6
Mar. 18, 1908	3.1	13.3	2.3
Mar. 24, 1908	3.9	18.1	33.2
Apr. 22, 1908	18.4	29.9	116.4
Apr. 29, 1908	8.0	19.0	5.8
May 20, 1908	7.9	20.6	...
May 27, 1908	8.1	23.3	...

In order to determine the time required to drive off a sufficient percentage of the light volatile constituents to render the tar satis-

Mr. Blanchard. factory for road work, a sample of the tar was heated for certain definite periods, the result of which was as follows:

			Total loss at end of period.
Loss at 120° after heating	½ hr.	3.6	3.6
Additional " " " " "	1 "	1.2	4.8
" " " " " "	2 "	0.9	5.7
" " " " " "	3 "	0.7	6.4
" " " " " "	4 "	0.7	7.1
" " " " " "	5 "	0.5	7.6
" " " " " "	6 "	0.5	8.1

As many failures of tar-macadam have resulted from the use of tar containing a large percentage of light volatile constituents, the State engineers, taking into consideration the foregoing analysis, have decided to require that all tar, before application, be heated for 2 hours at a temperature of not less than 150° fahr., the maximum quantity of tar heated in a 100-gal. kettle to be 50 gal. It will also be required that all water shall be removed from the surface of the tar in the kettle before heating. From one barrel used on the Narragansett section more than 2½ gal. of water were removed.

The pitch cost, f. o. b. Providence, $3.00 per bbl. of 30 gal. The freight from Providence to South Ferry cost $0.27 and the haul, averaging 2 000 ft., cost $0.10 per bbl., making the total cost $3.37 per bbl. The pitch used was medium hard, having a melting point at 105 to 110° fahr., and a specific gravity of 1.25. The volatilization analysis gave the following percentage results:

Volatile at 120° fahr.	None.
Additional loss at 212°–220°	1.73
" " " 240°–260°	1.53
Total loss " 260°	3.26

A Texas asphalt was used. It was purchased from the Texas Company, and cost, delivered at South Ferry, $21.50 per ton of 250 gal. The cost of the haul from the station to the road, an average distance of 2 000 ft., was $0.10 per bbl. of 42 gal. Therefore, the cost of the asphalt on the road was $0.09 per gal. This asphaltic product is listed by the Texas Company as Texaco Asphalt, Grade H. Analysis yields practically no volatile matter to 260° fahr., a melting point of 140° fahr., and only 8.8 gr. per gal. in a water solution.

The Tarvia was purchased from the Barrett Manufacturing Company, and cost, f. o. b. Boston, Mass., $3.25 per bbl. The freight to

South Ferry, a distance of 70 miles, over three lines, was \$1.26 per bbl. The barrels cost \$0.75 each. The rebate allowed was \$0.65, which, minus the cost of haul, and return freight of \$0.30 per bbl., made a net rebate of \$0.35 per bbl. The cost of the haul to the road, an average distance of 1 000 ft., was \$0.11 per bbl. The total cost was \$5.02 per bbl. Mr. Blanchard.

The results of the analysis of Tarvia A were as follows:

Specific gravity	1.182
Volatile at 120° fahr	2.4
Additional loss at 212°–220°	8.9
" " " 240°–260°	6.5
Total " " 260°	17.8

Having considered the cost and analyses of the materials used, the cost and details of the construction by various methods will be given.

The tar-macadam constructed by the mixing method required the services of two tar men at \$1.75 per day of 10 hours, and three common laborers at \$1.50 per day, in addition to the force usually employed on the No. 2 course. The cost of the labor, based on an average rate of progress of 233 sq. yd. per day, was \$0.035 per sq. yd. The cost of accessories, which would include rent, or interest and depreciation, on tar kettles, dippers, and cost of fuel, was \$0.005 per sq. yd. The saving by not using a watering cart, at \$4.00 per day, was \$0.013 per sq. yd. The cost of the tar, at 1.25 gal. per sq. yd., was \$0.093. The total cost of the tar-macadam, in excess of the ordinary macadam, was \$0.12 per sq. yd.

The difference in cost of the tar-macadam without the tar on the No. 1 course and with that tar (about $\frac{1}{3}$ gal. per sq. yd.) spread on the No. 1 course, was not appreciable. It is believed that the painting of the No. 1 course is not necessary. In common with all methods of construction, with the single exception of the Gladwell system, it is necessary, in order to secure a maximum penetration of the broken stone by the tar, and adequate incorporation of the tar in the macadam, to allow the No. 2 course to remain without rolling and sanding for from 1 to 3 days, depending on the climatic conditions. It was found to be inadvisable to roll the tarred surface during the warm part of the day, as there was a tendency for the No. 2 course to shift if the tar were soft.

In the construction of the tar-macadam by the penetration method, the tar was spread over the surface by dippers. This method was very unsatisfactory, an unequal application being the result. In order to procure an efficient road, more tar was applied in patching, the original application of 1.25 gal. being thus increased to 1.87 gal. The

Mr. Blanchard. penetration secured varied from 1 in. to 2½ in. If this method is to be used, pouring pots with fan-shaped spouts, or a fan-nozzle connected with a hose from a tank-wagon, should be used, or preferably a spreading machine similar to the Laissailly or Aiken. Even with a machine of the most approved type, and with the stone heated either before or after deposition, it is doubtful if the tar-macadam surface thus constructed would be as uniformly bound together as when laid by the mixing method. The average rate of progress on this section was 389 sq. yd. per day. The cost of the labor (which consisted of two tar men and one common laborer) was $0.013 per sq. yd. The tar cost $0.138 per sq. yd. Adding the cost of accessories and deducting the rebate due to not watering, the total cost was $0.143 per sq. yd.

In the construction of tar-macadam by the Gladwell system, the bituminous mastic, consisting of tar and stone chips varying in size from ⅛ to ½ in. in their longest dimensions, was mixed in a regular mortar box. This mixture was spread to a depth of ¾ in., and the No. 2 course was then laid upon it. A coating of tar was spread on the surface, and, after screenings had been applied, the section was thoroughly rolled. The upward penetration of the tar was not measurable, and the surface coat did not penetrate more than 1½ in. In order to procure satisfactory results, it will be necessary to have the No. 1 course so thoroughly compacted as to hold a semi-fluid mixture; the stone composing the No. 2 course should be larger than that generally used, and should be well heated, and, finally, it will be necessary to use a light asphalt roller in order to draw the fluid mixture gradually to the surface, and not attempt to crush the No. 2 course into the binder. Under no circumstances is it believed that the method will prove as efficacious or economical as either the mixing or penetration methods of construction. The rate of progress of this class of work was slow, and would average 156 sq. yd. per day. The labor item was high, two tar men and four common laborers being required, making the labor cost $0.06 per sq. yd. The tar, 1 gal. per sq. yd. in the mastic and 1.25 gal. on the surface, cost $0.167 per sq. yd. Adding the cost of the accessories and deducting the cost of watering gives a total cost of $0.22 per sq. yd.

Tar-pitch-macadam was constructed with and without using chips mixed with the No. 2 course. With hand-mixing, the omission of the chips is preferred, as the tarred chips have a tendency to coagulate, hence preventing a uniform mixture. The pitch and tar were used in the proportion of 1 to 3. The resulting surface was excellent, and could naturally be rolled and finished more expeditiously than tar-macadam. Under certain circumstances, as for instance, on a road subjected to more or less traffic during construction, this desideratum would be a great advantage. On the other hand, the proportion of heavy oils is less than when only tar is used, thus reducing the elas-

ticity of the surface and increasing the danger of cracking in cold weather. The rate of progress and the cost of labor were the same as for tar-macadam constructed by the mixing method. The total cost was $0.142 per sq. yd., including $0.035 for labor, $0.079 for 1.07 gal. of tar, $0.036 for 0.32 gal. of pitch, and the usual accessories charge and watering rebate. Mr. Blanchard.

In the construction of the tar-asphalt-macadam, 50% tar and 50% asphalt were used, or 0.625 gal. of each per sq. yd. The resulting road was ideal, from the standpoint of construction and as a finished product. A daily average of 233 sq. yd. was built, making the labor item $0.035 per sq. yd. The tar cost $0.047 per sq. yd., and the asphalt $0.056 per sq. yd. The total cost, including the accessories charge of $0.008 per sq. yd., and deducting the cost of watering, was $0.133 per sq. yd.

The asphalt-macadam, considered from an economical standpoint, was not successful. The primary difficulty was that it was impossible to obtain a thorough mixture of the asphalt and the stone, coagulation taking place at once. The resulting surface was entirely satisfactory, and could probably be built economically by using a suitable mixing machine and heating the stone. The labor and material items on this work were excessive, the cost of labor being $0.083 per sq. yd., while the cost of the 3.59 gal. of asphalt used per square yard was $0.323. The average rate of progress per day was only 117 sq. yd. The accessories charge was $0.008, and the watering rebate $0.013, making a total cost of $0.401 per sq. yd.

The Tarvia-macadam constructed by the mixing method appeared to be a *fac-simile* of the tar-macadam made with tar distilled for 3 hours on the road. It is believed that it is primarily a question of economics whether it is preferable to take gas-house coal-tar direct from the works and distil it on the road, or purchase distilled coal-tar, in the form of Tarvia, for example. It should be borne in mind, also, that tar distilled at permanent works will give a more uniform product. The rate of progress was the same as with tar-macadam, hence all cost items were identical, with the exception of the cost of 1.25 gal. of Tarvia used per square yard which was $0.124. The total cost of the Tarvia-macadam, therefore, was $0.151 per sq. yd.

The preservation of the surface of existing macadam roads has been attempted in Rhode Island by using the tar-painting method, and by the use of various oils as palliatives. One application of the tar painting has been used successfully in the preservation of the macadam surface for two years on a hill having a maximum grade of 7.28%, but, where it has been used on roads subjected to high-speeding motor-cars, parts of the coat of tar have invariably peeled off during the first season. On a town road in Peace Dale, painted with hot Tarvia by the usual method in August, 1907, barely 60% of the tar coat was

Mr. Blanchard. intact in March, 1908. The experience of France, England, and America with the tar-painting method has led many to the rational conclusion that it is not effective for macadam roads subjected to even normal motor-car traffic if high speeding is allowable.

Although the method has not yet been used in Rhode Island, the speaker believes that the most satisfactory and economical treatment of existing macadam roads is to tear up the surface to a depth of from 3 to 4 in. with a scarifier, re-shape the surface, and then rebuild as a tar-macadam road, using the penetration method of construction. On 1908 work in Rhode Island the cost of scarifying was \$0.007 per sq. yd. The cost of re-shaping, without the addition of new road metal, would be about \$0.005 per sq. yd.; hence the total cost, based on the cost of the penetration method used on the Narragansett Road, would be \$0.105 per sq. yd. If a tar-spreading machine were used, the cost, without doubt, would be reduced from 15 to 25 per cent.

The oil products used on the highways of Rhode Island include Dustoline, petroleum residuum, Ragland crude oil, petroleum fuel oil, and Texas asphaltic road oil. The average cost of oiling roads in Rhode Island, using about $\frac{1}{3}$ gal. per sq. yd., has been approximately \$0.01 per sq. yd. Experience would indicate that oiled roads, if efficient, should be treated at least three times during a season of eight months.

The speaker's conclusions, based on the maintenance work he has outlined, and the experience of French, English, and American engineers, are: First, that highways subjected to heavy high-speed motor-car traffic should be built with a bituminous macadam surface constructed by the mixing method; second, that existing macadam roads subjected to a similar traffic should be reconstructed as bituminous macadam roads using the penetration method, or, if re-surfacing with new road metal is required, by using the mixing method; and third, that the economical and efficient treatment of macadam roads subjected to a moderate amount of motor-car traffic is at present a matter of conjecture, requiring for elucidation the acquisition of reliable detailed information with reference to the use of the various palliatives now on the market.

Mr. Rablin. JOHN R. RABLIN, Esq.*—The roads built by the Metropolitan Park Commission of Massachusetts have been constructed from time to time between 1897 and the present, so that none is more than 10 years old. Heavy traffic is excluded from them, and only light pleasure travel allowed. Under these conditions, and with ordinary care, these roads would have been expected to require no extensive repairs up to the present, and they were generally in excellent condition up to the spring of 1906.

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At about that time the increasing use of the automobile became noticeable, and in July, 1906, these roads began to show the effects of this traffic. The macadam roads were stripped of their top surfaces, leaving the stone of the base entirely bare, and with no binder to hold it, the stone loosened, and the macadam began to break up. Mr. Rablin.

Investigation was at once begun to find a method of protecting the surfaces, and it was decided to treat, with a specially prepared coal-tar, portions of the roads showing the greatest damage. Consequently, in August and September, 1906, about 3½ miles, or 70 000 sq. yd., of macadam roadway were treated with this material, as an experiment.

The process of treatment was: First, all loose material was cleaned from the surface by sweeping with street sweepers and push-brooms, so that the tar might penetrate properly. The tar was then flowed upon the surface from tank wagons, in which it was brought from the works, at a temperature of about 200° fahr. It was spread with push-brooms, and allowed to set from 4 to 6 hours before covering. Stone screenings were then spread over the tarred surface in such quantities as were required to absorb the surplus tar which had not penetrated the macadam, and the road was then rolled once or twice, and was immediately opened to travel.

The cost of the work done in 1906, including all labor and material, averaged 6⅔ cents per sq. yd.

The results of the treatment, during the remainder of the year, were excellent, a new surface having been provided which could not be removed by the automobile travel, and the road was rendered practically dustless and entirely free from mud. No sprinkling with water was necessary.

This work passed through a very severe winter and, except a small section on which the sub-grade was poor, was in very good condition in the spring of 1907.

In July and August, 1907, it was necessary to re-treat about one-half of the 3½ miles treated in 1906, and to patch the other half. The average cost of maintenance of the whole 3½ miles of roadway, for 1907, was 3½ cents per sq. yd. No other repairs or sprinkling with water were necessary during the year.

During 1907, in addition to the maintenance of the work laid in 1906, various pieces of road, aggregating about 90 000 sq. yd. of roadway surface, were treated with this same material, at an average cost of 7.3 cents per sq. yd.

At the present time, June, 1908, work is in progress on repairs and re-treatment of the surfaces previously treated, and it seems evident that it will be necessary practically to re-treat once each year in order to maintain this work in good condition. Compared with the cost of re-surfacing the roadway with macadam, this cost is small, and the benefits are considerably greater.

Mr. Rablin. From observation it appears that the durability of this treatment is much greater and its life longer on steep grades and on roads with a good, well-drained sub-grade.

Generally, the results of the treatment of the parkway roads with this material have been satisfactory, and it has been very effective in the preservation of their surfaces and in making them practically dustless and free from mud. The treated surface has an appearance similar to tar-concrete, but is not as smooth. No odor is apparent after the material has been on the road a few hours, and what little there is at the time of application is not at all disagreeable.

Various experiments were also made in 1907 in the use of oil on the parkway roads. About 1 mile of gravel-surfaced road and $\frac{3}{4}$ mile of macadam-surfaced road were treated with a product of crude asphalt oil, from which the naphtha and other volatile substances had been removed, leaving the heavy base material of the oil with sufficient petrolene to make it fluid. This material was applied to the roads, without heating, and was allowed from 4 to 7 days for its proper penetration. Where surplus oil collected in any slight depressions or ruts in the road, enough sand or gravel was used to absorb it.

This treatment requires a road to be in particularly good condition, either new or newly re-surfaced, and very little cleaning is necessary, only the removal of loose dust.

These roads were treated with the oil in June, 1907, and have remained in excellent condition to the present time, a period of about 1 year. With some slight re-touching, and patching with the same material, arrangements for which are now being made, it is expected that these sections of roadway will remain in good condition for another year before requiring an entire re-treatment.

The cost of this work was 6 cents per sq. yd. for the material and its application, and the cost of maintenance for this year will probably be less than one-half that amount.

It has proved an effective treatment for both macadam and gravel-surfaced roads, and arrangements have been made for the treatment of about 4 miles, or 70 000 sq. yd. additional.

In 1907, about 12 miles, or 240 000 sq. yd., of macadam and gravel-surfaced parkway roads were treated with a mixture of water-gas tar and oil similar to that above described. This mixture was made in varying proportions of from 2 bbl. of oil and 6 bbl. of water-gas tar, to 4 of oil and 6 of water-gas tar. The greater the proportion of oil used, the longer the durability of the treatment.

This treatment is a dust layer, but is not particularly effective in the preservation or bonding of the road surfaces. It is, nevertheless, a good and economical treatment for gravel road surfaces where the bonding effects are not as necessary as upon macadam. It is most deserving of consideration on account of its cost, which is from 3 to $3\frac{1}{2}$

cents per sq. yd., and one treatment is practically effective for laying the dust for a whole season from May to November. Mr. Rablin

One other experiment is being tried at the present time, and that is the use, on macadam road surfaces which are in fairly good condition, of a mixture of water-gas tar and coal-tar in about equal parts. The use of water-gas tar in this and the previously described mixture, with oil, is for the purpose of diluting the heavier and more expensive material, and thereby reducing the cost per square yard for treatment, and also on account of its penetrating power. No cleaning is necessary for this latter treatment, except where an excessive amount of loose material may have collected upon the sides of the roadway. One or two days after application the surface is sanded lightly and the road opened to travel. The cost of this treatment is about 4½ cents per sq. yd.

It appears to be giving excellent results, and will doubtless prove very worthy of consideration for the preservation of roads which are in fairly good condition.

It has become absolutely necessary to adopt some one of the various forms of treatment to maintain macadam and other roads in fair condition where they are subjected to heavy automobile travel, and, although there are doubtless other, and possibly better, methods, those herein described have been proved to have considerable merit, particularly if applied to the road conditions to which they are best adapted.

Louis L. Tribus, M. Am. Soc. C. E. (by letter).—In the United States comparatively little attention has been given in the past to the proper maintenance of street pavements. Much attention has been given to the questions of construction, through all their features of subsoil, grade, drains, character of foundation, surface, cross and longitudinal slopes, relationship of pavement surface to traffic in reference to dust, cleanliness, slipperiness, durability, ease of repair, and cost of construction and maintenance. In general, cities have been content to make repairs when road surfaces have, through months or years of use and abuse, developed ruts or holes and a generally unsightly and even dangerous condition. This is not always the fault of the officials in direct charge, but of those who hold the purse strings, who are disinclined to appropriate money to maintain something that seems to be in fair order as far as hasty inspection goes. Mr. Tribus.

There can be no doubt in the minds of municipal engineers that the keeping of any form of pavement in perfect condition adds materially to its total life, and by adding together all costs of such maintenance repairs, for the normal reasonable life of the pavement, the total will not equal the costs of practically reconstructive repair, at longer intervals. If, in addition, there could be tabulated the expense in broken axles, strained tendons, paint and varnish injured by mud, clothing damaged, wear and tear on the nerves, profanity

Mr. Tribus. induced, etc., the balance would be very largely on the side of daily maintenance.

In that portion of New York City where the writer has more or less authority, there was inaugurated, some years ago, the system of daily inspection of all of the more heavily traveled roads, covering practically all classes of pavement, macadam, brick, asphalt block, sheet asphalt, wooden block, bituminous concrete, iron slag, granite, etc. In rainy weather, when depressions show most conspicuously, the men stationed on macadam roads, note the depressions and, if possible, at once clean them out and fill them in with new broken stone, of such assorted sizes as need may require; a little hand tamping, given at the time, finishes the repair, and a few days' traffic makes it indistinguishable from other parts of the roadway. When a road wears down so that the aggregate cost of daily repairing exceeds the economical limit, then reconstruction takes place.

What has just been said, of course, obtains more particularly for such a pavement as macadam than for the more permanent forms, though a neglected rut or an improperly made repair in any pavement, after opening the street for some purpose, not only causes an unsightly and even dangerous condition, but seriously injures the adjoining portions of the pavement and tends to lessen the life of the whole.

It has been the aim during these past years to make these repairs permanently and perfectly at the first day possible after an opening is made and back-filled. While such repairs can be more readily made in some classes of pavement than in others, the general principle involved is the same, and the result is almost as satisfactory in one as in the other.

Cleanliness is another very important factor in the life of a road of any class. A macadam road kept reasonably free from dust means freedom from mud; consequently, it saves annoyance, and the stone is held together longer and more effectively than is the case where a loose medium, like sand or mud, aids in breaking the bond and separating the stones.

The writer remembers a long stretch of macadam road, which actually wore down until the tops of the telford foundation stones showed, without any raveling of the surface, due entirely, in his opinion, to the freedom from dust and mud, the traffic being very heavy most of the time.

A sheet-asphalt pavement kept clean will last longer than when left in a dirty condition, as in the latter case water does not flow freely from the surface, and thus tends to rot the asphalt. If water can be kept from standing on the surface, the wear will come from traffic only. A block pavement kept clean is thereby freed from a great many disintegrating elements that find their way between the joints

and tend to wear the blocks on their edges as well as on their upper surfaces. Mr. Tribus.

It is a fair proposition, therefore, to say that a comparatively inexpensive pavement, well laid and kept clean and in constant repair, will outlast any more costly pavement that is neglected; and during their respective lifetimes one will give public satisfaction and the other will be a nuisance during all its later life.

The writer has only taken up this question of the cleaning of pavements as it affects the life of the pavement, entirely apart from the sanitary side and the general comfort of the community.

There are so many conditions affecting the selection of the form of pavement for any given street or locality, that it would make a lengthy discussion to go into that topic in detail.

Naturally, the first consideration is the ability of the community to pay for what are its best interests. Assuming, therefore, that the first cost is not limited, a pavement should be selected that will give the least noise, the smoothest surface, and may be cleaned to the best advantage, all consistent with the weight of traffic, and the traction power of horses, determined largely by the gradient.

With these general questions settled, then come the estimates for annual repairs; the total life of the pavement under the traffic estimated; inconvenience to the public by having the street out of commission for renewal, etc.

These conditions might readily indicate, therefore, the need for a more expensive and more permanent form than the first considerations would suggest; so that no general rule can be laid down for the original selection of a pavement. The principles of construction are standard, but the practice as to maintenance is far from being standardized; and the writer hopes such discussions as this may result in securing more continued and constant cleaning and repairing of all classes of pavement than is as yet customary in the United States, and he knows that the results will tend toward economy and comfort.

James Owen, M. Am. Soc. C. E.—The Society and the country at large should be congratulated on the excellent discussion presented by Mr. Blanchard. It is excellent in its arrangement, and in the description of work done. It also gives tangible results, for it shows the important item of cost, which should be impressed upon every engineer engaged in highway work so that his clients may have a basis of comparison in connection with the change which is about to come in road construction in America. Mr. Blanchard shows that the increased cost over old methods is 12 cents per sq. yd., and if that is impressed upon the people's minds, the small extra expense of building a road according to modern ideas and regulations will be quickly appreciated. Mr. Owen.

After much experience in road building, the speaker is compelled to confess that he must apologize for the work he has done. History

Mr. Owen. sometimes repeats itself, and it is a fact that about 1843, after the good roads era had been fairly completed in England, and they had the system which has been handed down, the construction of those good roads created a desire for self-propelled vehicles. They were not called automobiles then, but were steam carriages. They became so prevalent that the horsemen, and stage drivers, and all other persons not interested in that mode of transit, protested so vehemently that Parliament passed a bill abolishing the steam traffic on the highways of the country. This was really prior to the railroad era. After the lapse of a good many years, the self-propelled vehicle has come again, in a different form, and has come to stay, so that the engineer must now accept such means of locomotion as a fact and must make due provision therefor.

One point should be brought out here, namely, that, nowadays, the term "hard roads" should be eliminated. Hard roads, according to the popular idea, are not wanted. The later road developments show that a medium is being placed in the hard material to obviate the hardness, the abrasion, and the extreme wear, tear, and dust of those surfaces. The necessities for the surface to-day are what would be called pliability and resilience, a surface that will give and be elastic, and will be restored normally after it has done its work. It is proposed to introduce certain foreign material in order to encourage that resilience and elasticity. Two extreme cases may be cited to exemplify that idea. In California earth roads were covered with shale, and then sprinkled with oil, which gave a perfectly smooth surface and the desired degree of elasticity that prevents wear. The second extreme case is the ideal pavement made with asphalt. Both these surfaces are elastic, and do not wear out. Some years ago, the speaker asked one of the asphalt representatives how much wear there was on the surface of asphalt pavements. He replied that there was no wear. Careful examination shows that, practically, this is a fact. One section of a city road under the speaker's control, originally covered with macadam, had to be repaired, on account of travel, every two years, with a coating of about 6 in. of broken stone. Asphalt was substituted, and has been down about seven years, and there has been not a particle of wear, nor a movement of the surface in that time.

This expresses the speaker's idea of an elastic surface. If there is a perfect surface of this kind, the element of wear is almost eliminated. Furthermore, having that elastic surface, the question of the positive texture of the stone used is not of such great importance. New Jersey has been priding itself somewhat upon its roads. They are good because, for the system of construction in use, the material is good. The difficulty, in the different States and sections of the country, is in finding material with which to make good roads; thus this question of mixture is important, and may decrease the trouble of

obtaining for road surfaces such good material as was formerly required. The suburban districts of New York City have had the same experience in the treatment of roads as that related by Mr. Blanchard. Automobile traffic has affected the roads to some extent, and some method must be contrived to prevent the resultant damage. The trouble is similar to that in Rhode Island, and Mr. Blanchard has gone into the matter in great detail, and has given tangible information, and definite results. Mr. Owen.

The use of oil in the vicinity of New York City has not met with favor. It certainly eliminates the dust, but, in residence districts, the odor of the oil is very objectionable, and the oil dust that rises under certain conditions is injurious to everything it touches; consequently the results indicate that tar, or preparations of tar and asphalt, will be used finally to decrease the wear of the roads.

Three years ago the speaker re-coated an old surface with 3 in. of broken stone and tar, with the screenings on top, the cost being about 17 cents per sq. yd. The result was most satisfactory, the road having been neither sprinkled nor repaired since that time.

It seems proper here to allude to roads which are not within congested districts. Professor Baker, who is very enthusiastic about the improvement of roads in stoneless districts, where the necessity for the proper administration of the business of farmers is absolute, considers that the elimination of the necessity for extremely hard material is going to be an important factor in the future construction and development of roads in stoneless countries, like Nebraska, for instance.

The oiling of roads, without the intervention of other materials, seems to give satisfaction in certain localities. The plan of using inferior material and a binder with it, is one on which there has not yet been much experimentation, and the suggestion is made that, if possible, this matter be taken up for discussion, so that, in the development of roads for the Central States, some advantages may be gained.

Nelson P. Lewis, M. Am. Soc. C. E.—In the past the highway engineers of the United States have been disposed to confine their studies, investigations, and discussions to what have been called permanent pavements. By permanent pavements is meant those which are usually laid under contracts calling for a guaranty for maintenance for a year or a term of years, and in which the problem of maintaining and repairing is not assumed by the municipality or State almost immediately upon their completion. Mr. Lewis.

There has been a disposition to regard macadam pavement as a temporary expedient, in a city where the abutting property is not able to bear the cost of a more permanent road material, or as a mitigation of the discomfort of the unimproved suburban or country highway. Only in a few localities where there are numerous towns and villages have macadam roads been built and maintained so as to

Mr. Lewis. furnish highways upon which high-speed motor vehicles can be driven with any degree of comfort. It follows that the problems of constructing and maintaining roads of the macadam type have not been the subject of careful investigation by engineers, and the results of the work which they have done have not been presented in the careful manner which is demanded by the importance of the problem.

The engineering profession is greatly indebted to Mr. Blanchard for having placed before its members so full a description of the work of the Rhode Island State Board of Public Roads, and such a complete analysis of the cost.

Lately, special attention has been drawn to the construction and maintenance of such highways by the advent and general use of motor cars. As these vehicles pass along what have formerly been good highways they are followed by great clouds of dust, resulting in much discomfort to others using the highway and to those living in its immediate vicinity. A resident of Northern New Jersey recently said to the speaker that, while formerly one who wished a country home could purchase property 200 or 300 ft. in depth, place his house 100 ft. from the highway, and live in comfort, this has now become impossible. To avoid the dust raised by motor cars it is now necessary to locate one's dwelling 500 or 600 ft. from the highway.

The dust raised by motor vehicles, however, means more than physical discomfort. It means that the road material is being used up and wasted with great rapidity. The ordinary city pavement, especially bituminous pavement and possibly macadam, if kept in good surface, may be improved by the passage over it of heavy vehicles moving at moderate speed; and, within the limits of cities and incorporated towns, the speed of such vehicles is usually kept within reasonable limits by police regulation; but when automobiles, at speeds of from 25 to 50 miles an hour, pass over these highways, with broad rubber tires drawing the binding material from the road metal, the case is very different.

If these highways are not to be entirely destroyed, the speed of these vehicles must be kept within reasonable limits, and some method of construction must be devised which will result in a surface compacted with a binding material of such a character that it will not be drawn from between the pieces of road metal and carried away as dust, or special roads must be provided for high-speed motor traffic. If the latter plan were adopted, such highways could not be built at public expense, but would be the result of private enterprise, and the expense of construction and maintenance would be met by tolls paid by those using them. This plan has lately been adopted on Long Island, N. Y., where a motor parkway is being constructed for the exclusive use of high-speed motor vehicles. This parkway will be entirely separate from other highways, there being no grade crossings, and it is expected that the enterprise will be supported by the fees charged for its use.

Mr. Lewis.

To control the speed of motor vehicles on country roads is well-nigh impossible, and in most cases the only treatment which appears to be possible is that of a method of road construction and maintenance which will adapt the highways to the new conditions. This is the plan which is described by Mr. Blanchard. The objection which has been thought almost fatal to such a plan has been the greatly increased cost of construction, but the data submitted by Mr. Blanchard show, in a manner almost startling, that this increased cost, under an intelligent plan and competent supervision, would be relatively little.

Roads built with bituminous cementing materials may present exceptional difficulties in making emergency repairs. Breaks or holes in the surface will inevitably occur, either through a defective spot in the foundation or from some external cause. To repair such breaks, either by the mixing or penetration process, would be difficult and expensive under any system of maintenance, and the members of the Society would doubtless be glad to know whether Mr. Blanchard has formulated a plan for making such repairs. He states that the difference in cost between ordinary macadam pavement and the bituminous macadam described by him has been as little as 12 cents per sq. yd. This is certainly a remarkable record, and the cost of labor must be very low and its efficiency very high in Rhode Island. The speaker is under the impression that this slight difference in cost could scarcely be approximated in a typical American city, especially where the prevailing rate of wages and the minimum length of day are religiously observed. Still, the ratio between the ordinary contract price and the increased cost of the kind of construction described should be realized if equally efficient supervision and careful attention to detail could be insured.

Mr. Owen has spoken of the desirability of elasticity and resiliency in a roadway for motor cars, and has referred to the earth road as a type of the desirable highway. The speaker confesses his lack of faith in these qualities as desirable for an automobile highway for high-speed cars. At low speeds the earth roadway is admirable, but the speed at which cars run at the present day would seem to require a very hard and unyielding surface.

That the problem presented by the general use of high-speed motor vehicles has arrested the attention of the most painstaking highway engineers of the world is shown by the fact that the French Government has arranged for an International Highway Congress to be held in Paris during the present year for the express purpose of studying these problems. The French highways have been among the finest in the world, and it is stated that they are fast being destroyed by high-speed motor cars. The Congress is to consider, not only the problems of construction and maintenance of the roads, but that of cars and tires as well, with the use of chains and other devices to prevent skidding.

Mr. Lewis. The speaker has had the privilege of examining the Rhode Island roads with Mr. Blanchard, and has been greatly impressed with the results which have been secured. The surfaces of the bituminous macadam roads, constructed as he has described them, are practically perfect, although some of the roads have been in use for two years, and they are absolutely dustless. Portions of them are in sparsely settled localities where there is apparently no restriction upon the speed of motor vehicles, and motor cars pass along the roads swiftly and smoothly without the customary trail of dust, which, however, is emphatically and offensively apparent the moment the cars pass from the bituminous macadam to that of the ordinary type. In the speaker's judgment Mr. Blanchard has made notable progress in solving what is probably the most serious problem confronting highway engineers at the present time.

Mr. Houston. G. N. Houston, Assoc. M. Am. Soc. C. E.—The speaker had the good fortune to live during the first twenty years of his life in northern New Jersey, the section referred to by Mr. Owen, where excellent macadam and telford roads have been constructed from the crushed trap rock of the Orange Mountain. He is also somewhat familiar with the conditions in Illinois referred to by Professor Baker.

In Colorado many varying conditions are encountered. In the eastern part are the agricultural districts, where the farmer must be provided with roads over which to haul his produce to the nearest market or shipping point. Of this traffic, the most wearing on the roads is that caused by the hauling of sugar beets, the loads weighing from 2 to 4 tons.

In that part of the State good material for ballast is either lacking entirely or too expensive to obtain. On one of these roads, in the southeastern part of the State, the only material available was what appeared to be a fairly tough shale. On excavating to a depth of 12 in., however, it was found that the material below this point was only partly consolidated, being damp and pliable. On being exposed to the air for several days, it became quite hard, and similar to that on the surface. There being only a limited quantity of surface shale, it was decided to use the softer material below. This was excavated with slips, and placed on the roadway, 8 in. thick at the center and 2 in. thick at the sides, the roadbed being 15 ft. wide. It was then rolled to the proper cross-section with a 10-ton roller, and, after hardening, proved to be a satisfactory ballast. The cost, without the rolling, was 29 cents per lin. ft. of roadway.

In other parts of the State there is the problem of a very high water-table. In some parts of the San Luis Valley it is within 6 in. of the surface. Much of the soil there is a heavy adobe which absorbs this water by capillary action, freezes in the winter, and when thawing

FIG. 1.—TYPICAL REINFORCED CONCRETE BRIDGE FOR COLORADO ROADS.

FIG. 2.—PART OF THE RED MOUNTAIN ROAD FROM OURAY TO SILVERTON, COLORADO.

heaves the roadbed from 12 to 18 in. above grade. The only remedy for this seems to be to ditch the road on each side, using the material to grade the roadway from 18 in. to 2 ft. above the surrounding country. Mr. Houston

In the mountains there is usually excellent ballast, the best being a disintegrated granite. This resembles a finely crushed stone, being angular, and varying from $\frac{3}{4}$ to $\frac{1}{4}$ in. in greatest dimension. Some binder is usually needed with it.

When a mountain road in Colorado is properly constructed, it needs comparatively little maintenance, except when it is misused, as is often the case in the mining districts. For instance, a mining camp opens up, and roads must be built to get the ore out and supplies in. Not knowing whether or not it will develop into a permanent camp, the work is more or less temporary in character. This is more especially true of the bridges, which are usually rough log affairs. If a bridge goes out on account of high water, or becomes too rotten to carry the loaded wagons, and the district has not developed sufficiently to warrant a new bridge, the miners then begin to pack in and out, and it takes but a few trips of burros loaded with timber or rails to destroy the remainder of the road. Great damage to mountain roads is also caused by driving herds of cattle over them.

The Legislature of Colorado appropriates about $150 000 every two years for the construction of roads and bridges. This is used in making new roads, improving existing ones, and building bridges.

The grades exceed 20% on some of the old roads. On all new work under the State the maximum grade is limited to 12%, with a 6% maximum adverse grade. In the mountains the width varies from 8 to 14 ft., with a 16-ft. width at turnouts.

The law requires that the county in which a road or bridge is built by the State shall maintain it, so that the State, officially, has very little to do with keeping up the roads. In order to reduce this maintenance to a minimum, reinforced concrete bridges are built wherever practicable.

The State has completed, or has under contract, twelve of these bridges, varying from 462 ft. long (eleven 42-ft. spans) to a single-span, 125-ft. arch bridge. Most of these bridges, however, have been of the slab and girder type (maximum 50-ft. span, 16-ft. clear roadway), and designed for a 16-ton traction engine loading. These cost from $42 to $45 per lin. ft., including pile foundations.

Owing to the dry climate, the roads in Colorado are in excellent condition during 10 months of the year, and are seldom impassable on account of mud during the remaining 2 months. Owing to this fact, there is an increasing demand that grades be cut down to accommodate the growing automobile traffic.

During the present year (1908) the State has taken up the experi-

Mr. Houston. ment of convict labor on road building. A road has been planned through the State, extending from the northern to the southern boundary, and the convicts are now working at the southern end. Reinforced concrete culverts and bridges are being built entirely by this labor.

The design and supervision of construction of all State roads or bridges is under the State Engineer.

Mr. Wedgeworth. D. C. WEDGEWORTH, ESQ. (by letter.)—The importance of highway improvement depends not so much on its engineering elements, as on the fact that a real improvement adds a certain percentage to the value of the property it affects. The rural districts are beginning to understand that the slight increase in taxes, if spent on properly improved highways, is a good investment.

This work of improvement has been entrusted to the engineer, and it is safe to say that it has not proved a very simple problem. The usual method is to lay a macadam roadway. This has been to a large extent successful, but the writer believes that a slight alteration of the method will add to the life of the road and lessen the cost of construction as well as maintenance.

This cost is dependent on many things, mainly the source of supply of material, stone, filler, water, etc., and, unfortunately, the cost of improvement of a highway is a large factor in its treatment. Against increased cost may be balanced a cheaper construction with probably a poorer highway. For this reason the engineer must strike a fine balance between an ideal roadway at high cost and an inferior roadway at less cost. In arriving at a final result there should be no better criterion than the results obtained from methods and materials heretofore used.

Given a road having a porous yet stable soil, well drained, it is frequently found that a slight improvement of natural conditions affords a good roadway. The worst natural conditions require the best road, the most thought, and frequently the most expense. Under such conditions an ideal road may be prohibited by great expense.

The engineer is then forced to find the material and method of construction which will give the best road for the money to be expended; or, in other terms, the limit of expenditure is determined by the value of the results obtained per dollar of expenditure.

The basic principle of a macadam road is the placing of a suitable wearing surface over a solid base in such a way that the macadam will perform two functions, first, sustain the wear of traffic, and second, transmit to the base the loads imposed by the traffic.

Had the engineer always at hand—or the money to bring to hand—materials which would furnish the macadam capable of carrying out these functions, the problem would be more simple; but, to bring about these results with the materials at hand, calls for much thought, experience, and common sense.

Mr. Wedgeworth.

Although the matter of general location and sub-base of a roadway is very important, it does not enter into this presentation, as the writer wishes to deal only with the construction of the macadam proper. Having settled upon the location and prepared the base, or sub-base, as the case may be, the present method has for its object the formation of a 6-in. layer of crushed stone, bonded and placed in such a way as to form a solid mass impervious to water. To this end a stone of good binding qualities must be found. This must be bonded with a proper filler, and the whole consolidated by successive manipulations until this result is approached as nearly as may be. The roadway is then put on trial; if it stands, the material and manipulations have been correct; if not, reasons and repairs are in order. Considering the varieties of stone and filler at hand, differing with various roads, it is rather remarkable that so large a percentage of roads shows satisfactory results.

The point the writer wishes to bring out is this: The stability of the macadam is dependent on the bond, and this is dependent on the filler and the manipulation. Under the best conditions, there are seasons

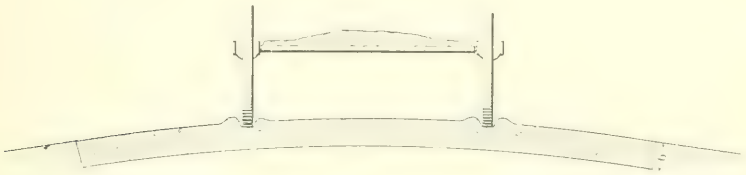

FIG. 1.

of the year when this bond is temporarily destroyed by the action of freezing and thawing. Heavy loads, also, may destroy the bond before the load is transmitted to the base. This fact is shown by the early repairs necessary on roads built under the present method. It is a fact, also, that when a country road is macadamized, not only will the number of vehicles be increased, but the weight of single loads will be doubled.

Fig. 1 illustrates this point. The stones are pushed up on each side of the wheel track, and not down into the base, simply because the bond is not sufficient to hold them together. A close examination during repair work will show that this is exactly the case.

Proposed Method.—Would it not be a more logical and common-sense way to place the stones in the macadam before crushing, as far as possible, than to break them up and try to bind them together again? Does a carpenter cut a post in sections with the intention of

Mr. Wedgeworth

splicing it again when in the building? Why not save this expense of crushing and rolling as much as may be?

Fig. 2 represents a section built after the following method: Having prepared the base in the usual manner, the larger stones are hauled upon the base and placed near one another, no special care being taken, as that would take much time and cause expense. These stones, in their vertical dimension, should average about 7 in. for an 8-in. thickness of macadam. Smaller stones, from the crusher, or spawls from the quarry, are then drawn in, and the spaces between the large stones well filled. A wearing surface of about 2 in. is placed over all, and the whole well rolled. As much stone dust as the stone will take up is then applied, and the surface is puddled, as in the present method.

The large stones transmit the load directly to the base. Filler is not depended on to bind the whole together, as the smaller stones between the large ones will do this. In fact, very little filler will reach the bottom, thus leaving it open for drainage, and avoiding the danger of disintegration by frost.

FIG. 2.

Comparative Cost.—To make an accurate comparison between the cost of the present and the proposed methods, similar conditions must be imposed on each. On roads where local field stone can be used without crushing, and crushed stone can be imported at reasonable rates, the percentage saved will be greater than where all stone must be hauled from the quarry in either case. Whatever the conditions, there is a saving of 50% of the crushing and 40% of the rolling in comparison with the present two-course road. Another point not to be overlooked is the utilization of the total product of the crusher. This method takes out the dust and 1-in. to 2-in. stone, putting all the remaining product in one bin. This product should be used to fill the spaces between the large stones, and the 1-in. to 2-in. stone for the wearing surface. In case a local stone is used for the body of the road, and trap is imported for the top, very little crushing on the ground will be necessary, and the whole product, with the exception of the dust, may be put in one bin and used to fill the spaces between the large stones.

On an average road, with field stone convenient to the roadway, and a good limestone quarry along the road, a comparison of cost would be somewhat as follows:

Mr. Wedgeworth.

COST PER 100 FT. OF 12-FT. ROADWAY.

			Present method.	Proposed method.
Preparation of base, say average of 6 in. excavation or fill, with necessary rolling and grading	22.2	cu. yd. at $0.75	$16.65	$16.65
Quarrying stone	23	" " " 0.50	11.50	
Quarrying stone	15	" " " 0.50		7.50
Hauling field stone	15	" " " 0.25 (in place)		3.75
Crushing quarry stone	23	" " " 0.25	5.75	
Crushing quarry stone	16	" " " 0.25		4.00
Hauling crushed stone	23	" " " 0.32	7.36	
Hauling crushed stone	16	" " " 0.32		5.12
Placing and rolling	133.3	sq. yd. " 0.15	20.00	
Placing and rolling	133.3	" " " 0.10		13.33
Comparative totals			$61.26	$50.35

These figures are not intended to fit any particular case, but are taken as an average from cost items on several roads. The first item will vary considerably, and will affect somewhat the total percentage saved under the proposed method. The cost of hauling field stone may vary, also, but, if stone walls are convenient, and if the stones are of moderate size or easily broken, the figure used may be reduced. The quantities used are, 23 cu. yd. of stone for the present method and 30 cu. yd. for the proposed method, giving a thickness of finished macadam of 6 in. in the former case and 8 in. in the latter. In the proposed method it is estimated that 50% of the volume will be crushed stone.

Where there are no field stones, but a good quarry is at hand, the estimate would be changed, as follows:

			Present method.	Proposed method.
Preparation of base			$16.65	$16.65
Quarrying stone	23	cu. yd. at $0.50	11.50	
Quarrying stone	30	" " " 0.50		15.00
Crushing stone	23	" " " 0.25	5.75	
Crushing stone	16	" " " 0.25		4.00
Hauling crushed stone	23	" " " 0.32	7.36	
Hauling crushed stone	16	" " " 0.32		5.12
Hauling quarry stone	15	" " " 0.32		4.80
Placing and rolling	133.3	sq. yd. " 0.15	20.00	
Placing and rolling	133.3	" " " 0.10		13.33
Comparative totals			$61.26	$58.90

Mr. Wedgeworth.

Under such conditions as abundant field stone with no quarry suitable for top stone, so that top stone would be imported in any case, trap might be specified for the top, and the estimate would be about as follows:

					Present method.	Proposed method.
Preparation of base					$16.65	$16.65
Hauling field stone	16	cu. yd.	at	$0.25	4.00	
Hauling field stone	22	" "	"	0.25		5.50
Crushing field stone	16	" "	"	0.25	4.00	
Crushing field stone	11	" "	"	0.25		2.75
Hauling crushed stone	16	" "	"	0.32	5.12	
Hauling crushed stone	11	" "	"	0.32		2.52
Trap, dumped in place	8	" "	"	3.50	28.00	28.00
Screenings (limestone)	5	" "	"	2.25	12.25	12.25
Placing and rolling	133.3	sq. yd.	"	0.15	20.00	
Placing and rolling	133.3	" "	"	0.10		13.33
Comparative totals					$90.02	$81.10

Although these figures cannot be exactly determinate, owing to inability to apply them, as they are, to any particular road, the writer believes that, as an average, they are fair, and that they indicate a reduction in cost and the production of a superior roadway, in favor of the proposed method.

It may be said, in opposition to this method, that it is impossible to roll the loose stone of such depth into the spaces among the large stones of the base. This, however, the writer does not believe to be the case, as the sub-base is being constructed by this method on many roads. Why not make the roadway itself a sub-base?

One often hears it said that macadam roads cost too much, both at first and in maintenance, yet contractors are slow to take them at appropriation figures. It is the duty of the engineer to furnish the best construction for the least money possible. The effort at improvement, however, has been directed toward making the present method more thorough and more expensive.

Macadam is being laid over comparatively good roads, built before the days of rollers and stone crushers. In the finer product let engineers not forget entirely the methods and means of those who built before them, for they built well and at little cost. More stone and less expensive manipulation is a step in the right direction. Try it.

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PAPERS AND DISCUSSIONS

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IRRIGATION.

An Informal Discussion at the Annual Convention, June 25th, 1908.*

By Messrs. George G. Anderson, F. H. Newell, J. P. A. Maignen, and A. L. Fellows.

George G. Anderson, M. Am. Soc. C. E.—The subject of irrigation presents such a wide field, is of such great importance and magnitude, and of such varied characteristics, that it cannot be treated otherwise than generally in the course of a discussion of ordinary limits. The speaker proposes, therefore, to confine his remarks to a particular field, to a section of Colorado which is typical of the whole State, which is, in large measure, typical of much that marks, or will mark, most other regions which will be reclaimed in the Western States. Mr. Anderson.

Near Denver may be seen some portions of a great agricultural territory which owes its present development solely to irrigation. The valley of the Cache la Poudre River is to-day as fine an agricultural territory devoted to ordinary farming products as can be found, not only in the arid region, but anywhere throughout the United States within the range of its possible products.

It represents the best that has been accomplished within the State of Colorado; what has been done there, can be and is being done throughout the State, and it has been done solely by private enterprise.

The work that has been achieved in that valley has really formed the basis for most other irrigation development, and not only has the practical result been the best and highest type of home building, which Mr. Newell has pointed out as being the ultimate aim of the governmental works of reclamation, but there has been built, out of

*Further discussion, had at the Convention, will be published subsequently, and additional written discussions are invited.

Mr. Anderson. the growth of the district, a code of laws which largely forms the foundation of all irrigation law throughout the arid region.

It appears to be difficult, even for the dweller in the arid region, to say nothing of the stranger within the gates, to realize and comprehend fully the significance of the agricultural development of this region and the future possibilities of similar districts under irrigation; and it is equally difficult to remember and acknowledge the debt owed from one generation to another, and from one region to another, in the process of such evolution as the arid region is now undergoing.

It will not be out of place if a very hasty sketch be given of the progress of irrigation development in Colorado. It has been marked by at least three periods, the individual, the community, and the corporate, and latterly the last has been in large measure resolved back, under recent legal enactments favoring such process, into what is really a form of the community effort.

The first or individual period was contemporaneous with, or followed closely upon, the advent of the gold seekers into the State. It is interesting to note that the dates of the earliest appropriations from the various streams on the eastern slope of the mountains run from 1859 to 1861—Denver was founded in 1858. Either the failure to find gold in paying quantities and the necessity of making a living in some form, or the ready money to be made in raising food for man and beast, and securing enormous prices therefor, drove certain of the pioneers to the less attractive but more certain pursuit of farming by irrigation. It is quite probable that these gained their knowledge of primitive irrigation methods from the Mormons, who undoubtedly introduced it into Utah in or about 1847.

The early irrigation farmers confined their efforts to the construction of small ditches for the reclamation of limited areas on the river bottoms, for the cultivation of hay and the cereals.

The next period was marked by what may be termed the "community" effort, which reached its best development in the Greeley Colony, in or about 1870. The community efforts sought the reclamation of more extended areas, by longer and larger ditches, built by somewhat improved methods, and as the result of the combination of several land owners.

In the latter Seventies and early Eighties the field of development attracted the attention of large corporations and combinations of capital, building larger ditches or canals, reaching out upon the mesas or bench lands, which, prior to this time, were considered unfit for cultivation and altogether inferior to the lands on the bottoms. This somewhat extraordinary view, in the light of later development, was quite extensively entertained of the original projection by ex-Governor Eaton, assisted by what was locally called the "English Company," of

the Larimer and Weld Canal, to irrigate the lands adjoining the now prosperous Town of Eaton. Mr. Anderson.

The corporate investment in large irrigation works, by no means confined to the Poudre River, continued throughout the decade from 1880 to 1890. Two companies, alone, invested, within that period, between $2 000 000 and $3 000 000 each.

About midway in this period, there came a realizing sense of the limited supply of water ordinarily obtainable from the streams, at least on the eastern slope of the mountains.

Under the law, no restriction was placed on the filing of appropriations upon any stream, and there was practically no real knowledge of the ordinary seasonal flow of any stream prior to that time. The earliest continued measurements of stream flow date from about 1885, though some gaugings were made as early as 1883.

It is not surprising, therefore, to find a canal with an appropriation of more than 450 cu. ft. per sec. dating from 1881, on a stream which averages about 840 cu. ft. per sec. in July, and with more than 4 000 cu. ft. per sec. appropriated ahead of it.

At about the same time, the Courts declared that the corporate companies, owners of canals, could not charge a price for water right, or "royalty," but that these canals were merely "common carriers"—that the act of appropriation involved the two separate acts of diversion from the stream and of application to beneficial use. Under such decision, the only revenue that could be derived from such canals was from rentals, and these were under the regulation of the Board of County Commissioners. While the Court subsequently held that these County Commissioners could not fix rentals so low as to be unremunerative, the immediate effect of the decision was disastrous to many of these corporate canal companies.

To that failure, the difficulty of securing at that time sufficiently rapid settlement of the lands tributary to the canal systems undoubtedly contributed. Nearly all farmers at that time were novices in the use of water for irrigation, and all the conditions tended to dissatisfaction and friction.

It is not an unusual experience to find the pioneers in any line of industry meeting with failure, which is finally turned to success by those who follow on the same lines. It has certainly been true that a large number of the unfortunate ventures in irrigation enterprise in Colorado have finally developed into successful undertakings. Many of them were projected before the conditions were ripe; they were merely in advance of the times.

The full realization of the "shortage of water," of the inadequacy of the stream to supply ordinarily all the canals and ditches depending on it, impelled the movement toward the storage of water.

Mention has been made of a canal with an appropriation dating in

Mr. Anderson. 1881, with more than 4 000 cu. ft. per sec. ahead of it, in point of time. It is rarely ever entitled to a supply of water from the direct flow of the stream. Finally realizing this, its owners built the first mountain reservoir on the Cache la Poudre, in 1885, and the storage of water for irrigation purposes had begun.

Since that time, that company has extended its operations to such an extent that it now owns or controls nine large reservoirs, containing more than 20 000 acre-ft., and provides water supply to them by the diversion of the waters of the Laramie and Grand Rivers across the main range. From the position of last on the stream for direct supply, the company has placed its system among the best and most successful in the valley, and more than 40 000 acres of the most productive land in the valley are dependent on it. This has been accomplished, not only by the diversion of water from other streams and by its storage and subsequent use, but also by the "exchange" of water, a system of management of water supply which has resulted in much practical benefit. In its simplest form, it means that a canal company which has stored water in basins below its canal level, and which it cannot again divert into its canal system, will deliver such stored water into another canal, on a lower level, the direct supply to which from the stream will be diverted by the canal company having the stored water.

As illustrative of what can be done in resuscitating an apparently dead venture, and in putting this "exchange of water" into practical and economical form, some reference may be made to the North Poudre Irrigation Canal. This canal was originally started in the early Eighties, and suffered temporary disaster about 1886.

The canal took its supply—merely flood waters—from the North Fork of the Poudre, and embraced about six reservoirs, with a total capacity of about 10 000 acre-ft. None of this was available for the lands tributary to the canal, and was only valuable for exchange. Mr. B. D. Sanborn, a well-known irrigationist, secured the system, and early in 1901 commenced the construction of the Fossil Creek Reservoir, which could impound the natural flow and flood waters of Fossil Creek, and could also be supplied from the flood waters of the Poudre River, by an inlet canal some 5 miles long. This reservoir is very remote from the other reservoirs of the system, and is on the opposite side of the river. It, also, was purely an "exchange water" proposition, and dependent on flood waters, and at its conception was met with the opposition of most of the best informed irrigationists in the valley, for several reasons.

The reservoir was built, however, notwithstanding considerable financial difficulty, and has proved a financial success. It is usually filled with water, and, as a result, the North Poudre ditch usually gets water through the irrigation season; and, largely on that account, the

fine section around Wellington has been developed, and raises the valuable late crops that have added so materially to the prosperity of the region. The reservoir has a capacity of about 11 000 acre-ft. Mr. Anderson.

Taking the reservoirs dependent on the Poudre River alone, there are more than twenty-five in operation, with a capacity aggregating about 100 000 acre-ft.

Some interesting details of these aids to irrigation are given in Bulletins 92 and 134, of the Office of Experiment Stations of the United States Department of Agriculture. In the latter, much consideration is given to the character and value of crops raised by the aid of these reservoirs, and this important statement is made (date 1903):

"The best estimates give $1 000 000 as the worth of these reservoirs to the Cache la Poudre Valley *each year.*"

In Mr. Tonge's "Handbook of Colorado Resources," the area of agricultural land in Weld County alone is given as 223 218 acres, with a valuation of $4 022 870.

In the introduction to Bulletin No. 92, dated December 15th, 1900, Mr. A. C. True says:

"In the Cache la Poudre Valley a series of reservoirs has been constructed by *local and individual enterprise*, which in connection with a system of rotation in the use of water, has resulted in doubling the area capable of being irrigated by the canals alone and in securing more thorough and effective irrigation than is usual without the aid of storage reservoirs. This account of the experience of the Cache la Poudre Valley in the construction and use of storage reservoirs has been prepared and is recommended for publication in the belief that it will prove valuable to many other localities, not only in Colorado but in other arid States."

And in his introduction to the same Bulletin, Colonel E. S. Nettleton, the author, the most prominent irrigation engineer in the early development of the canal system of Colorado, says:

"Ten years only have been required to build up the most complete system of storage in the United States. This has not been accomplished by a large expenditure of money, nor because of unusually favorable locations for reservoirs, nor was it wholly by accident that the people of this valley took up this line of irrigation development. Conditions forced them into it. They did not wait for the general government or the state to build reservoirs for them. * * * While this system is by no means completed, the owners have already reaped much larger rewards for their labors than they expected, reaching into the millions of dollars.

"Although the necessity for storage reservoirs and their utility and benefits are generally recognized, but few people outside of northern Colorado are aware of what has been accomplished in that locality by the utilization of small things or of the ingenuity displayed in making the most of them."

Mr. Anderson. And, in his conclusions, he makes the following remarks:

"Exchange of water facilitates its distribution and amplifies its use."

"The concert of action in the exchange of water appears to have lessened the number of legal conflicts."

and

"Greater economy in the application of water is attained by alternating rather than pro rating in times of scarcity, both among canals and among users from canals."

And he could well have added, what the experience of the years since he wrote the Bulletin has proved beyond peradventure, that the storage of water, by itself, in the sense of increased security to the water user and the certainty of its availability for application just when needed, has materially aided in the actual economy in the use of water itself, to which feature reference will be made later.

About 1896 Captain Chittenden, for the United States Government, examined, among others, two reservoir sites in the State of Colorado, both of which it might have been thought were beyond the reach of private enterprise. One was then called the Goose Creek Site, on the South Platte River; the other was Boyd Lake, tributary to the Big Thompson River.

The first was substantially constructed as the private enterprise of the Denver Union Water Company, and is now utilized for the storage of water for the domestic supply of the City of Denver.

Regarding the Boyd Lake enterprise, some interesting details may be given. A small section of it, known as the Seven Lakes, was developed some years ago as a storage reservoir for irrigation purposes as the private enterprise of Mr. B. D. Sanborn. In connection with this reservoir, Mr. Sanborn held some shares in two ditches which could feed it, and in an effort to store such ditch water, a litigation ensued which has far-reaching effects upon the irrigation interests of the State, and the speaker may be pardoned for a brief reference thereto.

It had previously been held by the Courts that, for flood waters and the winter flow of the stream, ditches with solely reservoir appropriations, or appropriations for the storage of water, had priority for filling over ditches with appropriations simply for direct supply.

This decision was used to prevent the Seven Lakes Reservoir from storing in the basin any of the water represented by its shares in the ditches during the irrigation season. The reservoir company did not claim the right to make any invasion on the river, but sought to have the right to change the character of farming from early crops, such as wheat and alfalfa, to late and more profitable crops, such as beets and potatoes, and to store in the reservoir in the early part of

the season—May, June, and July—their *pro rata* share of water in direct supply to use later in the season, and in this they were finally upheld by the Courts. Mr. Anderson

It has been charged that Court decisions in irrigation matters have been inconsistent, but consistency may mean stagnation, inconsistency may mean marching in the line of progress and development.

Seven Lakes Reservoir has a capacity of nearly 10 000, and can be developed to about 14 000 acre-ft. Boyd Lake is a very large basin, covering more than 2 000 acres at high-water line, with a depth of 68 ft. It is about 4 miles long and 1 mile wide, and its capacity is about 55 000 acre-ft.

There was much reason to believe it to be larger than any available water supply, but Mr. Sanborn has developed it, and in its first year there was stored in it more than 30 000 acre-ft. The waters stored are to be used in exchange also—and at present the reservoir is in the ownership of one individual, no arrangements having been completed for a reservoir company, and there is some consideration of utilizing it in connection with power development.

These developments are referred to as examples of the work of private enterprise prior to 1901. It is not intended that they should include all the work done by such, not even in the district specially considered.

It may be well to note the development of the Great Plains Reservoir system in the Arkansas Valley, with a capacity of about 264 000 acre-ft., as well as the Twin Lakes, the Sugar Loaf, and others.

The year 1901 has been mentioned, as that marks the period of another irrigation era in the districts under consideration. At about that time the introduction of the sugar beet and the demonstration of its adaptability to this district practically revolutionized conditions, in relation to water, to crops, and to values.

At about that time, also, some legislative enactments were passed which have given an impetus to further development in the construction of ditches and reservoirs.

The Carey Act, approved in 1894, empowers the Secretary of the Interior, with the approval of the President, to contract and agree to patent to States having desert lands, not to exceed 1 000 000 acres of such lands under certain conditions.

The Colorado Legislature, in 1895, accepted the conditions, and the grants of land to the State, under the provisions of the Act of Congress, and provided for the manner in which the irrigation, reclamation, occupation, and disposal of the same should be carried out.

The last Report of the State Engineer of Colorado states that three companies are taking advantage of this act, the largest making application for about 38 000 acres of land, with an estimated expenditure for works of more than $200 000.

Mr. Anderson. An Irrigation District Act was passed by the Colorado Legislature in 1901, and amended in 1905, and, under its provisions, several large undertakings have been completed, and a number are under way. The act gives such districts the power of quasi-municipal corporations, the right to construct or to purchase canals and reservoirs, to operate them, and to issue bonds for these purposes, such bonds constituting a lien upon the real estate within the district.

It practically provides municipal ownership, and by it the water users are enabled to combine for their own direct interest, and, with intelligent co-operation and management, the best results are likely to follow upon its extended adoption.

Through the energy mainly of one man, Mr. D. A. Camfield, the following districts have been organized, and the works completed since 1902:

	Acres.
Fort Morgan District	15 000
Bijou District	40 000
Green City	2 000
Julesburg	30 000
Riverside	60 000
Hillrose	20 000
Montezuma	55 000
	232 000

It is understood that more than $3 000 000 have been expended in the construction of these various enterprises.

In addition, the same gentleman and his associates have under way two districts, the Laramie Poudre, with 100 000 acres, and the North Sterling, with 80 000 acres, and an enterprise adjoining the City of Denver which will reclaim probably 60 000 acres.

Further, in the last Report of the State Engineer, there are three irrigation districts other than those already mentioned, in which bonds have been sold for $560 000 on 28 200 acres; and, since the date of that report, further bonds have been sold by the Otero Irrigation District for the construction of a reservoir on the upper Arkansas River.

By private enterprise, outside of the Carey and Irrigation District Acts, 50 000 additional acres in Weld County have been brought under irrigation by the extension of the Larimer and Weld Canal, the Greeley No. 2, and the Pierce Lateral, and by the extension of the Larimer County Canal; and there is also the very extensive reservoir undertaking in close proximity to Denver, known as Standly Reservoir.

It is also well known that other enterprises, mostly under the District Act, are in progress, the details of which it is not possible to give at this time.

It is not intended that this list should be exhaustive or attempt to

state the total expenditure contemplated, nor to give the present area now under cultivation by irrigation within the State, or the cost of the works which have made that possible—or the value of the reclaimed area. The only purpose is to indicate the extent, character, and value of the work which has been carried out and will be carried out by private enterprise. Mr. Anderson.

Within a reasonable time, and with reasonable conditions which are now apparently provided, it is not too much to say that private enterprise will undertake the conservation of all the waters on the eastern slope of the Rockies. There have been years, and those of moderate flood, in which all the waters of the Poudre River have been conserved and economically applied to beneficial purposes, and the steady extension of the enterprises in that valley indicate that ere long the maximum floods will be conserved in the same way. With the projects outlined on the South Platte, the same conditions will soon prevail in the immediate vicinity of Denver.

The various stages of irrigation development outlined at the outset inevitably call for some remarks on the character of the construction which then obtained. In the earliest days it was, naturally, of the crudest, the gradient of the ditch being developed either in the primitive Mexican way of allowing the water to find its course, or the bed of the ditch was laid out with the spirit level or triangle. Better methods obtained as the plans enlarged, but the critical observer in the Poudre Valley and elsewhere will still see parallel ditches, unnecessarily heavy gradients and distinctly bad alignment. It is not too much to say that these conditions are materially improving, that the engineer is permitted more scope, and that the projectors look for and demand a better character of construction throughout. Much more permanent work is being done than ever before, and the tendency is steadily toward betterment rather than otherwise, though the private investor looks for and expects marked economy in all lines, and the engineer has to regard carefully the returns on the expenditure.

In the past, the most serious error has been due to the projection and construction of large canals from rivers the normal water supply of which had previously been exhausted, and that error, not solely—and probably rarely—the engineer's, was due, as has been pointed out, to the lack of real information as to stream flow.

There has been a very marked improvement since 1887, but the best that is to be had to-day is far from being as complete or as accurate as is desirable. In view of the development of storage and of power plants, accurate knowledge is required of the flow of the stream for the whole season. At the present time, gaugings are ordinarily maintained for the irrigation season only.

The State authorities are crippled by the lack of funds to carry out the work properly and fully, and it is evidently impossible to

Mr. Anderson. arouse the legislators to make a sufficient increase of appropriations for this purpose.

Co-operation has been had at times with the hydrographic branch of the United States Geological Survey, and even that co-operation has been seriously threatened, as the funds at the disposal of the Survey are at all times inadequate for the work.

The importance of this matter to all engineers must be so apparent that it cannot require anything further than mere mention of the matter to secure the active assistance of the Society, individually and as a body, to secure, for the proper branch of the Federal Service, sufficient appropriations to maintain continued and accurate gaugings.

There is one gratifying condition in the present irrigation situation in Colorado, as compared with a generation ago, and that is the great progress which has been made in the use, and the economy in the use, of water by the user himself. It is not too much to say that the approach to a reasonable and low duty of water has been almost wholly due to the user himself, to the greater intelligence he displays in its use, and the sense of the resulting benefit to every interest affected. And that, again, is due to the greater reliance on storage, the greater sense of security storage conveys to the user, and to the sense of personal and pecuniary profit thereby.

The speaker trusts that in these remarks, hastily prepared as they have been, he has been able to bring eastern engineers into closer intimacy with the subject so vital to those living in the arid region, and to give local engineers an opportunity to discuss still further the subject that is ever old and ever new.

Mr. Newell. F. H. Newell, M. Am. Soc. C. E.—The object of water storage and distribution, for irrigation or for industrial purposes, is to make homes for citizens. This is the underlying consideration in the passage of all the Acts of Congress which have to do with the reclamation of the public domain, either by private, corporate or public funds. It is the basal reason urged by promoters or by public-spirited men who favor any reclamation project, large or small, or whether conceived for the purpose of making money or not. The argument used to attract general interest is that homes will be made, population increased, values will rise, and all corelated investments will be made more secure, because of the fact that these homes will be created and production will be stimulated.

The stability and permanence of the commonwealth are assured by the conservation of water resources which otherwise would go to waste. The destructive floods and the excess water of the streams are held in reservoirs. The stored waters when needed are put to beneficial use upon lands which without them would lie desolate. Men otherwise unemployed are put to work, and are enabled to become owners of homes. Water storage is thus one of the first and most

obvious of the many lines of conservation of the resources of the nation. Mr. Newell.

Conservation has come to be recognized at the beginning of this century as the great principle to be followed. Exploitation has characterized the last century. It has been carried to a point where serious-minded men are beginning to wonder whether it is not time to call a halt and take account of stock to see whether the resources of the nation are adequate, under the present wasteful methods, to realize the great promises which the future should hold in store. The President of the United States has taken up this matter. He has called a conference of the Governors of all the States, and this conference has gone upon record as declaring the firm conviction that conservation of our natural resources is a subject of transcendent importance which should engage unremittingly the attention of the nation, the States, and the people in earnest co-operation. This is a recognition of what has been long prominent in the minds of engineers; in fact, it may be said that this conservation movement has been initiated by the engineers who, from their training, have come to regard the conservation of waste as the great element of success. Our "captains of industry" have demonstrated this in their great combinations, where it is asserted that success is attained not so much by processes or invention as by the economies which are possible in operations on a large scale through the saving of the waste products and through the working up of the by-products into profitable output.

In considering, therefore, the question of water storage and distribution, we are simply keeping in line with the great movements of the present day, both in corporate and governmental affairs. The sure foundation upon which only success can be assured is a full knowledge of the facts. If we are intelligently to conserve any waste resources, we must acquire the most complete knowledge available concerning the matter in hand and all of the limiting circumstances. In taking up water storage, therefore, the first consideration is to obtain accurate facts concerning the topography of the country, its hydrographic relations, and the political or human elements, such as ownership of lands and of vested rights. We must obtain by actual observation and surveys on the ground all facts which are available, and even with all the physical data which we can obtain there must be a notable margin of uncertainty which can only be bridged by the use of sound judgment based on wide experience.

The lack of this full knowledge, so necessary for intelligent conservation of water, has wrecked many a brilliantly conceived enterprise. It is unnecessary to relate the details of any cases; the memory of every engineer who has reached middle life is stored with instances where enthusiasm has been substituted for knowledge, and good intentions have paved the way to bankruptcy. A recognition of this fact

Mr. Newell. has led the Government, in its effort at reclamation through water conservation, to go into preliminary examinations with a thoroughness which at times has seemed superfluous, but which, almost invariably, has been shown to be not only necessary, but perhaps has not been fully adequate for the solution of all the problems which have ultimately arisen.

The work of the Government in water storage, as a rule, has been preceded, as far as possible, by topographic surveys of the drainage area and of the lands which are to be irrigated. This has been accompanied by hydrographic surveys carried on by men of experience who, while the topographers were making maps, have measured the flow of water in the streams day by day and season after season, and have obtained all available information concerning the water supply from the time it reaches the ground until it disappears or has passed beyond control.

The methods of topographic and hydrographic surveying necessary for water conservation have been developed in accordance with western conditions. The United States Geological Survey was authorized in 1888 to investigate the extent to which the arid lands might be reclaimed, and there has grown up in the twenty years since that time a highly developed system by which results have been obtained economically and effectively. Both of these may now be regarded as highly specialized lines of work in which well-educated engineers are making a livelihood. Without the continuity of purpose and the broad field of work made possible by the Government, it would not be practicable to perfect the existing systems.

The information obtained by the topographic and hydrographic examinations of the Geological Survey has been utilized by the engineers of the Reclamation Service, in the preparation of designs for works, such as storage or diversion dams, head-gates, and canals suitable for holding the excess water or diverting it to lands where it can be used. There has thus arisen a need of experts in a field distinct from that of topographers and hydrographers, namely, men who have had experience in the storage and distribution of water, not merely from the engineering side, but also from the so-called practical or human side. There has thus been evolved by degrees the characteristic designing and constructing work of the Reclamation Service, with the subsequent requirements of maintenance and operation.

As we progress in the preparation of plans and reach the point where it is necessary to lay out systems of distributing water to the lands, a high degree of technical skill is not only requisite, but to this must be added for success a knowledge of human nature and of human institutions, laws, and court decisions. As we approach the place where the water is actually turned to the farm and distributed to the individual we meet conditions where the human element becomes more

and more prominent, and where the relations of man to man and to the community in general must modify very largely the practical considerations of the subject, overshadowing in part many of the simpler engineering details. Academic considerations may be fallacious, and may lead to the ruin of the best theoretical plans. More quarrels can arise in an arid region than on whiskey! There are to be considered, also, of course, important questions relating to the character of land and of products, nearness to markets, and a thousand other details which are not strictly engineering in themselves, and yet their successful handling makes or mars the value of the engineering work. Mr. Newell.

In all cases, whether the structures are built by public or by private funds, the works must ultimately pay for themselves. If built by investors, they must not only pay for themselves, but must yield a reasonable profit and interest upon the investment. If built by the National Government, under the terms of the Reclamation Act, the question of interest or profit does not enter directly, as water is furnished at the estimated cost, including in this the expenses of administration, which must be provided and must be considered carefully in any estimates of the cost of such works. Cost, therefore, is always an important element.

In the structures designed for the storage of water, the primary consideration beyond effectiveness is that of durability. These works must be among the most permanent built by Man, not only because their use will continue forever, but because of the fact that no risks should be taken where so many lives and so much property are involved. Plans for the construction of storage works, while they must be prepared with regard to reasonable economy, must be with a view to being not merely safe but looking safe. People must not merely be told that they are substantial, but when the plain citizen visits the work he must see for himself that there is every indication of the permanency and stability of a great storage dam. It will not do simply to tell him that the work is safe and that his children and his property, located beneath the dam, are in no danger; he must feel, to the very innermost recesses of his consciousness, that the structure is beyond all question.

Water storage may be provided in many ways. The simplest is that provided by Nature in the woods, where the litter on the forest floor holds back the water and serves to regulate the flow of the streams. Complete water storage must begin with the forests on the mountains, and any movement for full conservation of the waters of the country must begin with a full study of forest protection. The policies of the Federal Government have now been determined along this line. As far as can be done at this late date, the National Forests will be preserved for the conservation of the streams having their sources in the mountains of the West.

Mr. Newell. Next to forest conservation comes the consideration of engineering questions of increasing the storage capacity of natural lakes, or of making artificial lakes by closing narrow outlets through which the drainage from some basin of notable size escapes. Here is where begins the large work of the hydraulic engineer in the location, design, and construction of dams. Their character is various. In the vicinity of Denver are to be found almost all the types of dams which have been built by Man, from the rude timber and stone dam of the lumberman through the earth dams, rock dams, and finally concrete structures which characterize the present century.

The distributing works, as a rule, do not require such permanence of construction or massiveness of design as the storage works. They may be modified or rebuilt in accordance with the development of the country. There is required, however, a reasonable permanence in these structures, for around them are grouped all the improvements made by Man in an arid region. The irrigating canals soon serve as a sort of base line delimiting the desert from the irrigated farms. All property values are associated with the irrigating ditch, so that its first location and construction governs all future land values. A correct location may mean prosperity to a large section; an incorrect location may mean permanent loss to the community or otherwise valuable areas of land.

The speaker wishes to express his appreciation of Mr. Anderson's discussion, and to emphasize, if possible, some of the points which he makes, especially as to the position of the engineer in this great work of water storage and conservation. In the early days, as he has pointed out, each farmer could build his ditch according to his own ideas. With the development of the country and the realization that water is limited in quantity, while land is practically unlimited, has come the appreciation of the need of accurate knowledge of the water resources. There is thus a demand for accurate observation, and for skill to utilize the results of experience. In water storage there may be perhaps one of the combinations most dangerous to human life and property. Stored water, like fire, is an excellent servant but a bad master. The States of the West are recognizing this, and are creating or strengthening their State engineering departments, in order that they may supervise the construction of large storage works.

In ordinary construction, if mistakes are made they can usually be rectified, but when the public enters upon this great work of water storage, it must have structures which will outlast the centuries. There should be no possible risk to lives and property resulting either from the direct destruction of the reservoir, the lowlands, or from the indirect loss which follows the loss of the stored water. In this the need of good engineering is coming to be recognized more and more. This meeting in Denver will be of great aid to our professional

friends in the West in strengthening the hands of our local engineers so that they may demand larger recognition of the fact that the Engineer should dictate, rather than follow the desires of the promoter, or of the man who wishes to develop water. In the past the engineers have been compelled to cut too closely to fit the amount of money available. There is coming about a better recognition of the fact that adequate funds must be provided in advance, in order to build structures which will be reasonably permanent, and that this is as important as a full knowledge of the water supply which can be depended upon to fill storage reservoirs. Mr. Newell.

It is interesting to note that the use of temporary material, such as wood, is gradually being done away with in irrigation works, in favor of more permanent construction. This may be called the age of concrete, in Western irrigation development. Hydraulic cement and concrete are being used more and more; concrete flumes, concrete turn-outs, and especially concrete storage dams are being built. This demand for cement is being met by the building of cement mills in different parts of the West, and with the increasing demand there is a lower price. New supplies of raw material are constantly being found; there is hardly a large county in the West where materials are not found favorable for the making of some form of cement. There is danger, however, of going to the extreme. It has been found that some of the smaller concrete structures have failed, or crumbled, and it is necessary that investigations be made as to the cause of failure of some recent concrete work. Such failure is due, presumably, to the strong alkali waters coming in contact with the cement. Chemical reactions, presumably not yet fully understood, take place between the sulphates in the natural waters which are used to a certain extent in irrigation or occur in connection with drainage.

J. P. A. Maignen, Assoc. Am. Soc. C. E.—This discussion on irrigation has suggested to the speaker a few thoughts which may perhaps not be altogether devoid of interest. It has been stated that in some irrigation works, where the water is pumped from muddy rivers, and where the country to be irrigated is flat, it has been found advisable to construct near the head-gates of the works a settling basin or grit chamber of sufficient capacity to reduce the velocity of flow so as to assure the deposition of the heavy silt. Mr. Maignen.

Would it not be interesting to consider for a moment what is called "mud," and what harm or good it may do in irrigation works? It is evident that stone, gravel, and sand can do no good on the land, but, on the contrary, may do much mischief by accumulating in the channels intended to convey the water. On the other hand, the fine "mud," of organic or inorganic origin, which is light and capable of floating in moving water, is useful, partly to close up leaks in the earth channels and partly to fertilize the soil. Therefore, at all irriga-

Mr. Maignen. tion works taking water which is usually or occasionally muddy, it would appear to be desirable to provide some small works intended to effect the division of these two constitutents of mud—the heavy grit, which is to be prevented from getting into the irrigation system proper, and the light floating loam, which may be allowed to go wherever the water goes.

The second matter of interest is this: The water of rivers is generally soft and particularly suitable for irrigation, but what about ground-water which contains lime and magnesia in solution? The speaker well remembers the great interest taken in the question of water by the Jury on Horticulture at the Paris Exhibition in 1889. The members of that jury were practical men. They knew the difference between watering plants with soft water and with hard water. They knew that the latter chokes the stomata of the leaves, the H_2O is partly evaporated and partly absorbed, but the lime salts are left behind as a deposit, and, if seen through a powerful microscope, would appear like limestone stalagmites. They also knew that hard water applied to the ground to feed the roots is unable to take into solution the organic or inorganic substances which are necessary to the life of the plants, and which soft water easily dissolves. In other words, hard water brings about the anemia of delicate plants by closing the respiratory pores of the leaves and by coagulating or turning to stone the soluble food of the soil. The members of the jury were so much impressed by this fact that they were on the point of recommending the use of distilled water for watering orchids and other delicate house plants. When they found that hard well water could be softened and made practically equal to rain water by softening processes, they showered medals and commendations on the inventors.

The speaker supposes that the National Government, engaged in these great irrigation works, is fully alive to the desirability of having accurate information on the quality as well as on the quantity of water available, and upon its suitability in different regions for raising the various kinds of crops.

When in the East, and previously in England and France, the speaker has often heard it said that the waters of the West were "alkali," but he has not found any one yet able or willing to tell what is meant by "alkali." In England and France water is said to be "hard" or "soft." It is considered "hard" when it contains more than 85 parts per million of bicarbonate of lime, sulphate of lime, carbonate of magnesia, and sulphate of magnesia (counted as carbonate of lime); and "soft" when the total amount of dissolved mineral solids is less than 85 parts. The salts above named can be decomposed, precipitated, and separated from the water by water-softening processes, and this has been done extensively in Europe in order to make

hard water suitable for drinking and for boiler use, for hot-water systems, etc. Mr. Maignen.

Would it not be desirable that the chemists of the National and State Governments, if they have not already done so, should take up the study of "alkali" waters, and determine how far they are objectionable in agriculture, and whether anything can be done to correct them?

The third point suggested is: What is "sour" soil? It is said that a soil is "sour" when it refuses to grow plants. What is the cause of the sterility of such soil? Is it that the roots are "drowned" in water? Is it that the seeds or roots are eaten or decomposed by animalcules or micro-organisms growing in excessively wet soils? Or, is it that the soil itself has been poisoned by the toxines or soluble residue of the metabolism of the plants that have grown on it before, or by the toxines produced by the micro-organisms actually growing in the impoverished soil?

The first of these suppositions is that commonly accepted.

The second was suggested to the speaker by an observation made in the laboratory. Corn had been planted in muddy material with an excess of water. It did not grow, and upon microscopical examination it was found to have been reduced to a jelly-like paste, and worm-like animalcules were seen in proximity to the rotten corn. Was there any relation between the decayed grains and the animalcules?

The third suppositon is the object of scientific inquiry now being carried on by the chemists of the Bureau of Soils, United States Department of Agriculture, at Washington, D. C.

If any member can give any information on the subject of "sour" soil, it will surely prove of considerable interest to all.

Another subject may be mentioned. In filtering water and sewage it has been found, over and over again, that coke is an excellent filtering medium, but it has always been deemed too expensive for large plants. Hence it is that sand, gravel, and cinders have been generally chosen. It has occurred to the speaker that if coke is the best material for the purpose, it should be used, and if the cost is too high, that cost should be reduced by finding a new use for the coke after it has done duty in filters.

In these days, when everyone is concerned in the preservation of the "natural resources" of the country, and when the land is being impoverished for want of fertilizing material, why should the mud collected on filters be washed away and returned to the river? Should it not be collected and distributed on the land with care?

Has coke any value as a manure? It should be remembered that the word "manure" comes from the French word, "*manœuvrer*," which means "working by hand" or "tilling." Upon inquiry, the speaker was encouraged to study this feature, and he thinks that he can answer in the affirmative. He believes that new coke has a value,

Mr. Maignen. as a manure and as a soil sweetener, at least equal to its cost, and that when it is charged with "mud" from water filters or sewage plants, it has a value in excess of its cost.

"It improves the physical condition of the soil, together with its water-holding capacity, particularly on heavy clays."

By helping to darken the soil, it "makes it more absorptive of the sun's rays and therefore warmer."

New coke contains 56% of cellular space. This cellular space absorbs water and gases in a concentrated form, and gives them up later as required. If to these natural qualities are added the nitrogenous matters derived from the solid residue of water or sewage filtration, the result is a product likely to be of some value to agriculture.

The speaker has used coke in several filter plants, but more particularly in what is known as the Belmont Preliminary Filter at Philadelphia, dealing with 40 000 000 gal. of water daily. Experiments made with the spent coke of this filter plant have satisfied him of the correctness of the above views.

The question of the color of the soil is exceedingly interesting. In some of the books on agriculture may be found the statement that dark soils may be 14° warmer than light soils. One day, the temperature being below 40°, the speaker put his hand on a tuft of black top soil which had been exposed to the rays of the sun for a few hours. The earthy material warmed his hand. Coming back to the same spot a few hours later, the speaker suggested to a friend that he should put his hand on a piece of coal which lay on the ground. The coal warmed his hand. He turned the piece of coal upside down, and found that the under surface was warmer than the upper surface, and very much warmer than the surrounding atmosphere.

Many have heard of the experiment made by Benjamin Franklin. He put a black cloth on a piece of ice and the sun's rays melted the ice quickly. He put a white cloth on another piece of ice under the same conditions, and the ice did not melt for a long time. This evidently shows that black absorbs the sun's rays more than white.

Another observation made by the speaker may be worthy of record at this time. At the filter plant designed by him for Lancaster, Pa., which has now been working for some years, a somewhat curious phenomenon was observed which may give room for thought and discussion.

The filter plant is composed of fifteen beds, 16 ft. wide and 150 ft. long, and filled with sand. On seven of the beds a coating of fine coke had been deposited on the surface of the sand. The coating was not more than $\frac{1}{16}$ in. thick, and the coke material used was as fine as lampblack—the surface looked black. Seven of the other beds had no coke, the sand being exposed to the sun's rays. There was

the same depth of water above the sand in all the filters—about 18 in. The plant was put in service at the end of April. The filter beds were uncovered, so that the full sunlight could act on the water. After a few days, the seven beds which had no coke were literally covered with a luxuriant growth of algæ, like a forest of diminutive pine trees; the whole surface was green, but there was not the slightest trace of algæ or green in the seven beds which had the coke. The coke of some of the sterile beds was scratched in spots so that some of the sand was exposed to the action of the light; in the next day or two algæ grew over the exposed sand spots and not where the coke film was intact. Mr. Maignen.

What caused the difference? The water was the same, the sand was the same, the exposure was the same, the only difference being the film of coke. Were the coke filters sterile because the black material absorbed the sun's rays, while the exposed sand in the other filters reflected them? The speaker can offer no reasonable explanation. He simply states the fact for further investigation.

Another phenomenon of the same nature occurred at South Bethlehem, Pa. In a filtered-water basin, the walls were made of rip-rap. These stones had become black from the soot of passing locomotives. The superintendent of the water-works, thinking that he would show off the beauty of the filtered water, whitewashed the stones above and below the water level. A few days afterward a beautiful crop of green algæ grew on the whitewashed stones which were under the water, and gave some anxiety lest they should ultimately give a bad taste to the filtered water. After a certain time the soot of the locomotives blackened the stones again, the whitewash disappeared and the algæ disappeared also.

It has been stated, over and over again, that it is necessary to cover reservoirs containing filtered water to protect it from algæ growth. The speaker is not satisfied that this is absolutely necessary. Algæ do not grow in water which is dirty or full of suspended clay, because it may be supposed that the suspended matter prevents the transmission of light, but they grow profusely in clear water.

It has been seen that black appears to be a protection against the growth of algæ. The speaker, therefore, would suggest that further observations be made along these lines. Perhaps it may be found that a coat of black asphaltum paint, applied to the walls and the floor of a reservoir, will protect the water against the growth of algæ. This, of course, would be more economical than roofing. The same kind of experiment might be made with the concrete portion of irrigation works.

A. L. Fellows, Esq.—The conservation and use of water is a subject in which the engineers of the West have more interest than those of the East. In arid countries, water is the most valuable asset. The West, therefore, is interested in the greatest possible extension and best possible use of its water supply—in fact living depends on it; Mr. Fellows.

Mr. Fellows.

agriculture, stock interests, and mining industries are to a very great extent dependent on its proper conservation, and the important question naturally arises: What is the best use that can be made of such supply? The problem is a very complicated one. The speaker has considered carefully the discussions on this subject, particularly those by Mr. Anderson and Mr. Follett, each of whom presents many important matters. There are one or two points, however, which the speaker will emphasize to some extent, and also some suggestions which he will add.

A few years ago, a great engineer, Major J. W. Powell, a master in the subject of the conservation and use of water supply, devoted much study to this great question. At the time that he made his well-known recommendations, he had probably studied the subject more thoroughly than any other engineer, particularly with regard to the use of the waters of the West. It was his opinion that the greatest benefit would be derived by allowing the water to be used as many times as possible. For example—taking the Arkansas River as an illustration—he said that its waters should be stored high up in the mountains, for the development of electric power, for mining, for lumber mills, for stamp mills, for the lighting of cities, for transportation lines, and the other innumerable uses of hydro-electric power. Then the water should be stored again in other reservoirs, and passed on down to the plains, where it would serve for irrigation and other agricultural uses. Major Powell said to a Congressional Committee that the waters of the Arkansas should be used where they could be used to the best advantage, which was along the cañon of the Arkansas, about Cañon City, and again about Pueblo; then, by storage on the side streams, they could be used again, for example, in the vicinity of such a valley as that of Rocky Ford, and so on down the river. The same conditions apply to a great many Colorado streams; for example, on the South Platte River there are numerous reservoir sites which may be utilized in the development of hydro-electric power in the mountains, and the water released from these reservoirs and again stored in others may be used to as great advantage, or even greater, in irrigation below.

The situation is greatly complicated, however, by the fact that the priorities of the irrigation ditches—the irrigation rights—are superior, to some extent, in time at least, to the later claims for power purposes; but there are certain features which must be remembered. Water is an asset of such great value in Colorado, and is so indispensable to its prosperity, that it must have the widest and best possible use. The western streams furnish a water supply greater than required by the irrigable lands under them; therefore, in time, their waters will be carried across the range to a much greater extent than they have been, because they will be more valuable on the eastern than on the western slope.

Attention is called to a suggestion by Mr. Follett in regard to the

Mr. Fellows.

use of the waters of the Colorado River. In coming centuries the best results in the use of the waters of this river will undoubtedly be obtained by storing them in great reservoirs in the Rocky Mountains, and subsequently equalizing the flow of the Grand, the Green, the Gunnison, the San Juan and other great streams that make up the Colorado. Power will be developed as the result of such storage; the flow of the streams, after being equalized through these reservoirs, will be used in the irrigation of great tracts of land such as those in the vicinity of Grand Junction; then the water will be stored in other reservoirs, perhaps in the cañons of the Green and of the Grand, and then it will be used in the Imperial Valley in the way described by Mr. Follett. The same may be done on the Rio Grande. It is true that there was considerable early use of water on the Rio Grande, but in this, as in other things, "the old order changeth, yielding place to new," and, on this river, also, there must be, in time, the best and most extended use of the water that is possible. It is a natural law, and cannot be repealed. There is a higher type of civilization in the more temperate climates, and there the water will be used to the best advantage.

Another example is the San Luis Valley. The waters, eventually, will be stored in the mountains surrounding this valley, and be used in that locality. There are other important reasons why such an asset as this should be used to the best advantage. The water in such localities will go farther, and will serve more ground; in other words, its duty will be greater, more lands can be irrigated, and, besides, there is the return water from seepage which can be re-utilized.

It is a suprising fact that fully one-third, or in some cases one-half, of the total quantity of water used in irrigation returns to the stream. On the Black River of New Mexico, for example, half the water used for irrigation returns to the stream and is used again. It is clear that such re-use of water would not be possible if it were required that it must be used lower down on the river at the expense of the lands above. The South Platte might be cited as another example, the return waters being about one-third of the total quantity used. If, for example, all the waters of this stream were allowed to flow to the plains of Nebraska, owing to some early priority there, involving necessarily great losses in transit, as well as the impossibility of securing the advantages of return seepage, there would be a tremendous loss to Colorado. The same is true of nearly all the streams of the West, for most of them rise either in Colorado or in Wyoming. Thus, on the Grand River, there is a great surplus of water going to waste. In the development of Colorado the best possible use of water depends on turning at least a portion of it over to the eastern side of the range. Objections will be made, but it has been done, and it will be done again and again, until that point is reached where the water supply shall have been most fully conserved.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE IRVING BAILEY, M. Am. Soc. C. E.*

DIED MARCH 28TH, 1908.

George Irving Bailey was of English and Holland Dutch descent. Dr. Solomon Bailey, his grandfather, was for many years a prominent physician in Bethlehem, Albany County, New York. Dr. James S. Bailey, of Albany, his father, was one of nine children, and was graduated at the Albany Medical College in 1853; he practised in Alabama and Texas, and was a surgeon in the Confederate Army between 1861 and 1866, after which time he practised in Albany, New York, until his death, July 1st, 1883. Dr. James S. Bailey was married to Frances J. Keith, of Augusta, Georgia. The subject of this memoir was a great-great grandson of John Keith, a colonial soldier who fought in the Battle of Lexington.

George I. Bailey was born at Hempstead, Texas, on December 24th, 1861, and was graduated at the Albany High School in the class of 1880.

He entered the Department of the State Engineer and Surveyor in September, 1880, was a rodman up to November, 1884, then leveler on construction until November, 1885, then draftsman in the Department, and was appointed Assistant Engineer in Charge in September, 1887, and so continued until June, 1892. As Assistant Engineer, he had charge of the lengthening of a number of the locks in the Erie Canal, designing and making plans for and building the culverts, waste-weirs, retaining walls, and iron bridges along the line of the State's canals, and making miscellaneous surveys for the State, and defending suits against it.

He was in charge of the survey for the storage dam across the Genesee River, and conducted tests for the tensile and crushing strength of concrete in connection with this dam. These tests were among the earliest of this character. He was appointed Superintendent of Water-Works of Albany, New York, in June, 1892. On May 16th, 1892, a new Board of Water Commissioners had been appointed, and the late Elnathan Sweet, M. Am. Soc. C. E., former State Engineer and Surveyor, having been chosen as its President, requested Mr. Bailey to sever his connection with the State work and accept this office. Mr. Bailey held the position of Superintendent of Water-Works in Albany until February, 1902, and while in that office some of the most valuable work of his life was accomplished, the complete

* Memoir prepared by Horace Andrews, M. Am. Soc. C. E.

record of which is fortunately preserved in his annual reports, which are models of painstaking application.

He was at first engaged with his routine duties, but, at the same time, was deeply interested in the proposed new water supply from Kinderhook Creek, and attended to much of the work necessary in developing the scheme. This plan, however, was denied support, the necessary legislative action being withheld, and, acting under a new Board of Water Commissioners which supplanted that of which Mr. Sweet was President, he assumed charge of the installation of new pumps for increasing the supply of raw water from the river by 15 000 000 gal. per day. In connection with this work, he planned and supervised the laying of 11 200 ft. of 30-in. main. He continued his interest in the Kinderhook Creek development, which he hoped might supplant the use of the river water, and supervised the continued weir measurements for that source of supply, reporting to the new Commission, on January 1st, 1895, in regard to these weir measurements, as follows:

"An examination of these records will show that, notwithstanding the protracted and severe droughts of the summer, all claims made as to the sufficiency of the creek as a means of supply have been sustained."

In Mr. Bailey's fifth report, dated January 30th, 1897, the matter of the filtration of the river water was first discussed by him. The city was at this time hopelessly involved in the continued use of the water of the Hudson River as its chief source of supply, and it was apparent that filtration was the only resource for improving the unsanitary condition of the water. Mr. Bailey made personal visits to various filtration plants, and the matter was finally submitted to Allen Hazen, M. Am. Soc. C. E., in consequence of whose recommendations the slow sand filters were adopted in preference to mechanical filters with chemical precipitation of solids, which had strong advocates. The slow sand filter project was recommended to the Common Council on February 13th, 1897, the estimated cost being $478 000. The plans were approved on April 19th, of the same year, and Mr. Hazen was appointed Chief Engineer, and, under the joint supervision of Mr. Hazen, Mr. Bailey, and the Water Commissioners, the filters were successfully completed.

Mr. Bailey's seventh, eighth, ninth, and tenth reports are filled with valuable descriptions of the construction of the filters, and with accounts of their operation and of the consequent decrease in the typhoid death-rate in the City of Albany. These reports, relating to the development of the filtration works—which were regarded as models, and visited from all parts of the United States—are of the highest value to the engineering profession. They are profusely illustrated, and, in their entire preparation, show Mr. Bailey's interest and unremitting care.

Upon the cessation of Mr. Bailey's superintendency of the Albany Water-Works, he undertook work for the development of an ice plant near Albany, utilizing the overflow of the Maezlandt Kill, one of the oldest sources of Albany's water supply. The ice-collecting ponds planned by Mr. Bailey have been in successful use since their completion by him in the summer of 1902. The upper pond has a concrete dam 30 ft. in height, while the lower pond is formed by the construction of an extensive dam of earth with a maximum height of about 18 ft.

Late in the fall of 1902, Mr. Bailey was called to New York City to undertake the completion of some defaulted contracts in the Borough of the Bronx. These were administered so satisfactorily to his employers that his relations with them continued to be of a cordial nature until his death. Mr. Bailey resolved to continue the business of a contracting and consulting engineer in the Borough of the Bronx and did so with great success until the close of his career. He secured contracts and successfully completed in that Borough the regulating, grading, curbing and flagging of many streets, avenues and roads. He also constructed sewers in Boston Road and in Bryant Avenue. At the time of his death, he had under way, and about one-third completed, a water filtration plant in Yonkers, New York.

In 1898, Mr. Bailey married Mrs. Eva Fish (née Williams), of Utica, New York. Four children were born to them: William Weaver, Lois, Helen, and Catharine.

While apparently in perfect health, he was stricken with heart trouble at his home in the Bronx on the afternoon of Friday, March 27th, 1908, and died the next morning before his brother, Dr. Theodore P. Bailey, could reach him from Albany. He is survived by his widow and his four children, by his mother, one brother, Dr. Theodore P. Bailey; two sisters, Mrs. John R. Kaley and Miss Corrine Bailey, all of Albany, New York.

Mr. Bailey was elected a Member of the American Society of Civil Engineers on October 1st, 1890. He joined the New England Water Works Association on December 14th, 1892, and continued his connection with that Association until December 31st, 1906. He was a member of the Masonic fraternity, having attained the distinction of the thirty-second degree. He was elected a member of the Albany Club on January 28th, 1893, acted as its Secretary from May 1st, 1901, to May 1st, 1903, and resigned on October 1st, 1903. During the past few years he was a member of the Contractors' Association, in the Bronx, and was active in its development. He was also a member of various social organizations in New York City.

Mr. Bailey's contributions to engineering literature are entirely in connection with his water-supply work. In addition to the valuable reports comprised in the volumes of municipal records of the City of

Albany, he wrote several communications for *Engineering News*. His discussion* of Mr. Hazen's paper, "The Albany Water Filtration Plant," relates to the operation of the filters. Mr. Bailey contributed to the *Journal* of the New England Water Works Association an article concerning the care of fire hydrants in winter. His article entitled: "The Effect of Water Meters on Water Consumption in the Larger Cities of the United States," was reprinted in the *Journal* of the New England Water Works Association in June, 1901, the editor stating that the paper contained so much valuable information as to warrant its republication without apology. At the 1901 Convention of the American Water Works Association Mr. Bailey presented a paper on "The Albany Filtration Plant, and Some of the Results Obtained."

Mr. Bailey was a diligent and persevering worker, and obtained the success that comes from painstaking application. His friends testify to his personal worth: C. A. Crane, Assoc. M. Am. Soc. C. E., writes: "Mr. Bailey was universally esteemed by all the contractors and city officials with whom he came in contact to an unusual degree," and this writer closes by saying: "thanking you for the opportunity to do what little I may in memory of one for whom I felt the greatest affection."

Chapman L. Johnson, M. Am. Soc. C. E., a friend of long standing, says:

"He was one of my best and dearest friends, and the news of his death is a sad and severe shock to me. Mr. Bailey was a man of indomitable energy and will power and of the strictest integrity. His success was due to persistent insistence on theories arrived at and from inflexible integrity of motive."

Mr. Hazen, who was so long his associate, adds: "Mr. Bailey was a big-hearted man, a good friend, and one who could always be counted on."

RICHARD MILFORD BERRIAN, M. Am. Soc. C. E.†

DIED DECEMBER 23D, 1907.

Richard Milford Berrian was born in New York City on May 17th, 1850. He was educated in the public and private schools of that city, his attention being given principally to such studies as would fit him for engineering work. In 1869, at the age of nineteen, he entered the employ of the New York and Harlem Railroad Company as draftsman in the Engineering Department under the late Isaac Craig Buckhout, M. Am. Soc. C. E., who was at that time Superintendent and Chief Engineer of the Harlem Railroad. He was promoted in

* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 296; an abstract of this discussion may be found in the *Journal* of the New England Water Works Association, Vol. 14, p. 329.

† Memoir prepared by S. L. F. Deyo and F. S. Curtis, Members, Am. Soc. C. E.

1871 to the position of Assistant Engineer in charge of the construction of terminals at the Grand Central Station, and, in the following year, was made Division Engineer in charge of a portion of the improvement of the Harlem Line on Fourth Avenue between Forty-second Street and the Harlem River, in New York City, for the elimination of grade crossings and the construction of four tracks.

This position Mr. Berrian held until the completion of the work in 1876, from which time until 1890 he had supervision of several important works, notably the survey and construction of the Manhattan Beach Railroad, as Assistant Engineer; the Boston, Hoosac Tunnel, and Western Railway, as Assistant and Chief Engineer, and as Division Engineer for the location and construction of the Buffalo Division of the New York, Lackawanna, and Western Railroad.

From 1890 to 1907 he was continuously in the employ of the New York, New Haven, and Hartford Railroad Company in charge of numerous large and expensive works, among which are the improvements through South Norwalk and Bridgeport, the construction of four-track draw-bridges across the Connecticut River, Pequonnock River, and Norwalk River, the six-track lift bridge across the Fort Point Channel at the South Terminal, Boston, Massachusetts, and at the time of his death he was in charge, as Chief Engineer, of the construction of the bridge over the Taunton Great River at Fall River, under the direction of a joint committee composed of the Railroad Commissioners, the Harbor and Land Commissioners of Massachusetts, and the County Commissioners of Bristol County.

Mr. Berrian was one of the ablest engineers of his day. An expert draftsman, his designs were definite, accurate and easily understood. His modesty in all professional matters was such that his ability was only known to those with whom he came in personal contact.

He was successful in all his undertakings, putting his whole strength and time into the work that he had in hand, and, although he was very exacting in requiring his plans and orders to be carried out promptly and accurately, he was quick to recognize merit and to give full credit to all his employees for their work, commanding their respect and that of his employers.

Mr. Berrian was elected a Member of the American Society of Civil Engineers on May 4th, 1887.

JOHN EDWIN EARLEY, M. Am. Soc. C. E.*

DIED DECEMBER 20TH, 1907.

John Edwin Earley was born at Walpole, Massachusetts, on December 31st, 1842, and was of Puritan stock, his father being John Earley, and his mother Sarah Otis, of Scituate, Massachusetts. He

* Memoir prepared by Francis Webster Blackford, M. Am. Soc. C. E.

was fitted for college at Phillips (Exeter) Academy, and was graduated from Union College, in the class of 1867, with the degree of Civil Engineer. His studies at college were interrupted for nearly two years by service in the Union Army, in the War of the Rebellion, as a volunteer in the 16th Massachusetts Battery, which was a part of the Army of the Potomac. His army service was from March, 1864, until he was mustered out, in July, 1865.

After graduation, and until 1873, Mr. Earley was employed as Assistant Engineer on the construction of the Portland and Ogdensburg Railway, the Johnston and Glenerville Railway, and in other engineering work in New England.

In 1873 he went to the Cincinnati Southern Railway as Resident Engineer in charge of the first residency, Division "E," and remained there until the completion of the work, in 1876. His residency covered the work from Somerset, Kentucky, to the south side of the Cumberland River, some of the heaviest and most difficult work on that railway, which was noted for the excellency and permanency of its construction, especially of the masonry work. This work was a distinct departure from the ordinary railway practice in America, and elicited favorable comment from European technical journals of the period.

For a time after 1876, Mr. Earley was in the Government employ, on the improvement of the Tennessee River near Florence, Alabama.

From 1879 to the summer of 1881, he was engaged on the surveys of the Atchison, Topeka and Santa Fé Railway, in New Mexico and Arizona, being at times harassed by hostile Indians and most of the time working under the protection of a military escort furnished by the United States Government.

In the summer of 1881 Mr. Earley crossed the line into Mexico, and thereafter, and until the time of his death, played an important part in the development of that Republic, occupying many positions of importance and trust, among which were the following:

In 1881-82, he was Principal Assistant Engineer to the late W. R. Morley, M. Am. Soc. C. E., Chief Engineer of the Sonora Railway. From November, 1882, to April, 1883, he was Principal Assistant Engineer to Lewis Kingman, M. Am. Soc. C. E., then, as he is now, Chief Engineer of the Mexican Central Railway. From 1883 to 1887 Mr. Earley's work was near Guadalajara, Mexico, first as Chief Engineer of the surveys and construction work in the rough country lying between there and Tepic, described by the late A. M. Wellington, M. Am. Soc. C. E., in his "Economic Theory of the Location of Railways"; and in surveying and building the Guadalajara Branch of the Mexican Central, of which he was Chief Engineer.

Subsequently, Mr. Earley was Chief Engineer of the Mexican Southern Railway, and located this road from Puebla to Oaxaca through a very rough and difficult country; the road, however, was built by an English construction company under its own engineer.

He was also for a time Chief Engineer of the Cuernavaca Railway during its construction from the City of Mexico to the Balsas River, and later had charge of improvements and changes in the line of the Interoceanic Railway between Mexico City and Vera Cruz.

After 1902 Mr. Earley was engaged in a general engineering practice, with an office in the City of Mexico, and, at the time of his death, was Consulting Engineer and a Director of the Tlahualillo Cotton Company and also of the Guerrero Iron and Timber Company, which is developing large tracts of mineral and timber lands in the State of Guerrero, Republic of Mexico.

At Auburn, Maine, on January 15th, 1873, he married Miss Etta Sawyer, daughter of James Sawyer. This was followed by thirty-five years of congenial companionship, for Mrs. Earley accompanied her husband wherever it was practicable for a woman to go, and shared the hardships and wanderings incident to the lives of so many engineers. She survives him.

Mr. Earley was Chairman of the Entertainment Committee at the Annual Convention of the American Society of Civil Engineers in the City of Mexico in July, 1907, and soon after that meeting went to the United States, where he spent several months among his old friends and in the haunts of his boyhood in Massachusetts. This was a pleasant finale to a long and useful, and more or less arduous, life; for, on returning to Mexico, while the steamer was lying in the Port of Progreso, Yucatán, he suffered a stroke of paralysis from which he never recovered, and from the effects of which he died a few months later at the beautiful home in the City of Mexico which his industry had created and his good taste embellished with many evidences of refinement and culture.

Mr. Earley was a man of sterling character, and an engineer of large experience and good judgment, of which his work bears evidence. He had a cordial, sociable, generous disposition, and was an entertaining talker. His reminiscences of early experiences in the Western countries and in Mexico could always command attentive listeners.

Mr. Earley was elected a Member of the American Society of Civil Engineers on April 15th, 1876. He was also a member of the Masonic Fraternity and the Grand Army of the Republic.

OTHNIEL FOSTER NICHOLS, M. Am. Soc. C. E.*

DIED FEBRUARY 4TH, 1908.

The shock which is invariably felt at the tidings of the death of an active man in full health and vigor was experienced by the many friends of Othniel Foster Nichols when they learned that his life had gone out instantly and without warning in the early morning of Febru-

* Memoir prepared by Nelson P. Lewis, M. Am. Soc. C. E.

ary 4th, 1908, at his home, No. 42 Gates Avenue, Brooklyn, New York. All the day before he had been actively engaged in his professional duties at the Department of Bridges, had lunched with a group of his friends at the "Engineers' Table" at the Astor House, when he was as jovial and affable as was his wont.

He was descended from Rhode Island families which had taken a prominent and patriotic part in the affairs of the State, and whose ancestors settled in the Rhode Island Colony early in the seventeenth century. The son of Thomas Pitman and Lydia Foster Nichols, he was born at Newport, Rhode Island, on July 29th, 1845. Early in his life the family moved to Brooklyn, New York, where he attended the public schools, and in 1862 entered a machine shop, where he was employed for several years before beginning his professional training at the Rensselaer Polytechnic Institute, from which he was graduated with the degree of Civil Engineer in 1868. This practical and theoretical training, with a capacity for persistent and faithful work, made him a resourceful man who could readily adapt himself to any conditions. Immediately after graduation he was employed in the development of Prospect Park, Brooklyn.

Mr. Nichols took part in building the first elevated railroad in New York City, and taught mathematics in the evening course at Cooper Union, and in 1870 became a member of the engineering staff of Cooper, Hewitt and Company. In 1871 he went to Peru, where he was engaged for four years on the difficult work of locating and constructing the Chimbote and Huaraz and the Lima and Oroya Railways. Returning to the United States in 1876, he became Assistant Engineer and Superintendent of the Edgemoor Bridge Company in the contract of that Company for building the Sixth Avenue Elevated Railway in New York. Owing to delays in this work, caused by injunctions, he joined the engineering staff of the Department of Parks, which had charge of the work of developing the annexed district, now the Borough of the Bronx.

In 1878 he again went to South America, as Resident Engineer of the Madeira and Mamoré Railroad, in Brazil. This project ended in disaster, and Mr. Nichols was obliged to go to London in connection with litigation growing out of it. In 1879 he became Assistant Engineer in the bridge shops of the New Jersey Steel and Iron Company, and in 1882 he became Resident Engineer on the Louisville and Nashville bridge across the Ohio River at Henderson, Kentucky. Upon the completion of this structure, in 1886, he became Chief Engineer of the water-works of Westerly, Rhode Island, but after a short time he resigned to take part again in the development of the Elevated Railway system of New York City, this time as Principal Assistant Engineer of the Suburban Rapid Transit Company in building what is now the Third Avenue line in the Borough of the Bronx. In 1888 he became

Chief Engineer of the Brooklyn Elevated Railroad Company, in which capacity he designed and built a large part of the elevated lines belonging to that company, while in 1892 he became General Manager of the Company. When the "New East River Bridge Commission" was organized, in 1895, and Leffert L. Buck, M. Am. Soc. C. E., became Chief Engineer, Mr. Nichols was appointed Principal Assistant Engineer, and with this great structure, now known as the Williamsburg Bridge, he was closely identified from its beginning. In 1904 he became Chief Engineer of the Department of Bridges, which office he held for two years, and from 1906 until the abrupt end of his life he was Consulting Engineer of the same Department. He was therefore closely identified with the design and construction of all the great bridges across the East River, except the Brooklyn Bridge.

As this brief outline will show, Mr. Nichols' professional life was one of unusual activity, and, while his experience was varied, his time and energy were devoted chiefly to structural work, and during his later years to some of the world's greatest bridges.

Mr. Nichols showed keen interest and took an active part in professional organizations other than this Society. He became a Member of the Institution of Civil Engineers of Great Britain on February 2d, 1892, and in 1897 the Institution awarded him the Telford premium for his paper, "The Brooklyn Elevated Railway." He became a Member of the American Society of Mechanical Engineers in 1896. He was a Member of the Brooklyn Engineers' Club, and its President in 1904, a Member of The Municipal Engineers of the City of New York, and its first Vice-President. He was, at the time of his death and for some years previously, President of the Engineering Department of the Brooklyn Institute of Arts and Sciences. He was also a member of the Engineers' Club of New York, of the Crescent Athletic Club, and of the Municipal Club of Brooklyn, a trustee of the Polytechnic Institute of Brooklyn, and a member of the Vestry of St. Luke's Protestant Episcopal Church of Brooklyn. He was always active in the Alumni Association of the Rensselaer Polytechnic Institute, and in 1895 delivered the address to the graduating class of that institution.

Well read in the best literature, and an observant traveler, he was a forceful writer and speaker, and as a companion he was entertaining and delightful, with a great fund of anecdote and reminiscence drawn from extensive travel and large acquaintance. Men of great professional attainments and of mental power are often lacking in those human qualities which make them loved as well as honored; but such qualities Mr. Nichols possessed in an unusual degree. A keen sense of honor in his business and professional relations; an unassumed pleasure in the success of his friends, and a warm sympathy in their reverses; a readiness to help those who needed assistance, not only by

advice and good counsel drawn from his broad experience, but by giving of his substance; to his friends an unswerving loyalty which did not stop to consider his own personal interests—these are the qualities which made for him a host of friends, each one of whom felt a deep sense of personal loss when they heard of his untimely death.

It is not often that a man of mature years who has attained high professional standing shows a live interest in young men just beginning their life work. Many students and young engineers found in Mr. Nichols a ready, sympathetic, and patient listener to whom they could confide their perplexities and their disappointments, their hopes and ambitions, without danger of cynical comment or caustic criticism.

His active, useful, unselfish life, full of buoyant hope, and enriched by a rare capacity for the enjoyment of friendship and association with his fellows, came to its close without waning of power or impairment of vigor, and left a fine example of honor, loyalty and rectitude.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on June 7th, 1876, was always active in the affairs of the Society, and served as a member of the Board of Direction in 1892 and 1893.

GEORGE EDWARD THOMAS, M. Am. Soc. C. E.*

DIED MARCH 6TH, 1908.

George Edward Thomas, born September 21st, 1841, in Haverfordwest, South Wales, Great Britain, was the son of John and Elizabeth (Walters) Thomas, and one of the youngest of ten children, seven girls and three boys. He received his early education in the Grammar School of Haverfordwest and the National School of Pembroke Dock. In July, 1856, he entered Her Majesty's Academy, Pembroke Dock Navy Yard, and, upon his graduation, served under Oliver Lang and John Inman Fincham, remaining until the spring of 1870.

He showed such marked energy and ability as a youth that, when resigning from Her Majesty's service, his commanding officer at first refused to grant his release.

On October 1st, 1864, Mr. Thomas married Miss Elizabeth Anne Jones, of Pembroke Dock; and, of their five children, two daughters and a son survive.

In July, 1870, Mr. Thomas moved to Chicago, Illinois, where he made his home until 1901, though his varied professional interests were largely in other fields.

From 1870 to 1877 he was with the Illinois Central Railroad under Mr. L. H. Clark, Chief Engineer. During this time he had charge of the construction of the superstructure for the first 3 000 ft. of breakwater for the Government Harbor, Chicago, and also the lake-

* Memoir prepared by the Secretary from material furnished by Mr. Thomas' family.

shore protection for the Illinois Central Railroad. Later, he put in the cradles and fitted out the transfer boats for that railroad at Cairo, Illinois.

In 1877 Mr. Thomas entered the employ of William Sooy Smith, M. Am. Soc. C. E., a pioneer in the use of pneumatic caissons for foundations, and had charge of building and placing pneumatic caissons for a bridge over the Missouri River, at Glasgow, Missouri, for the Chicago and Alton Railroad.

In 1878 he was with Mr. C. E. Cooley on work for the Federal Government on the Missouri River at Nebraska City, Nebraska.

In 1879 he had charge of building and sinking pneumatic caissons for a bridge over the Missouri River at Plattsmouth, Nebraska, for the Burlington and Missouri River Railroad.

In 1880 he was called in on a bridge over the Red River in Arkansas for the St. Louis, Iron Mountain and Southern Railway.

In the fall of 1881 he was with General William Sooy Smith building a bridge for the Charleston and Savannah Railway at Hardyville, South Carolina. Later, he put in the foundations for a bridge over the Yazoo River, near Vicksburg, Mississippi.

Early in 1883 Mr. Thomas had charge of the installation of machinery for the Railroad Appliance Exposition in Chicago, of which the late Mr. John McGregor Adams, of the Adams and Westlake Company, of Chicago, was President. Later in the same year Mr. Thomas built the first electric railroad for traffic at the Louisville, Kentucky, Exposition, for Mr. Adams, this road having been an exhibit at the Railway Appliance Exposition in Chicago. Many professional men of the present day will remember the pioneer engine, the old "Judge." Comparing this first power-house of his construction with the great power-house on 59th Street, New York City, of which he was the Supervising Engineer, Mr. Thomas once wrote:

"The horse-power developed at Louisville, required to run the first electric railroad, equalled fifty; the horse-power developed at the 59th St. Power-House equals 142 000. The 50 horse-power at Louisville represented all the then used electrical power for street railroad purposes, and that at the 59th St. Power-House, not one-half the amount required in the City of New York; all this change and improvement in just twenty-one years."

In later years, apropos of Mr. Thomas' appointment as Supervising Engineer on this great New York City Subway power-house, Mr. Adams wrote:

"Am much pleased to know of the place that has sought you. It is a great compliment to your ability and skill, and I am satisfied no one could fill it more satisfactorily than you."

In 1883-84 Mr. Thomas had charge of building and sinking the pneumatic caissons used in the construction of the bridge over the Susquehanna River at Havre de Grace, Maryland, for the Baltimore and Ohio Railroad, and, for that railroad, he built and sunk the pneumatic caissons and had full charge of work on a bridge over the Schuylkill River at Philadelphia, Pennsylvania.

His many years of practical experience in subaqueous work made him expert in that branch of his profession. Concerning their mutual relations, General Sooy Smith writes:

"During the many years of his association with us, he occupied very important and difficult positions, and discharged his whole duty, in every one of them, with rare fidelity and ability, contributing largely to our success in the great works we carried out with his valuable assistance. He was indefatigable in his diligence, and his ingenuity and skill made him equal to every emergency."

In 1886 Mr. Thomas had charge of and sunk the shafts and drifted 300 ft. on the Canadian side, and 100 ft. on the American side of the St. Clair River at Port Huron, Michigan, for a preliminary tunnel under the river for the Grand Trunk Railroad.

He had charge of, built, and sunk the pneumatic caissons for the Huntington system, over the Ohio River at Cincinnati, Ohio, in 1887, and also, later, made borings for the proposed bridge at Vallejo Junction, California, for the same system.

In August, 1888, Mr. Thomas assumed full charge of sinking a shaft for the Chapin Mining Company, at Iron Mountain, Michigan, for the Sooysmith-Poetsch Freezing Company. It was the first shaft sunk by the Poetsch method in the United States, and was brought to a successful issue in the early spring of 1889.

Charles Sooysmith, M. Am. Soc. C. E., writes, regarding their many years of intimacy:

"His courage was an inspiration that pushed many a great undertaking to speedy completion, and always helped those below and those with him to work with spirit and happiness."

In 1889-90 Mr. Thomas had charge of the construction of the Lubec Narrows Light House, at Lubec, Maine. This, together with the Shubenacadie Bridge at South Maitland, Nova Scotia, he considered as among his most difficult works, both being located on the Bay of Fundy.

Quoting Mr. Thomas, in a discussion of his paper "Piers of Midland Railway Bridge, Shubenacadie River, South Maitland, Nova Scotia,"*

"I made all the borings and all the surveys in New York harbor from Cortlandt street to Ft. Tompkins and up the Kill von Kull, for the proposed tunneling, etc., which is to be done under New York City."

This was for the Roberts-Corbin Syndicate, in 1892.

* *Journal*, Western Society of Engineers, Vol. VI, 1901, p. 172.

In 1892-93 Mr. Thomas had charge of, and sunk part of the caissons for the New Central Bridge over the Harlem River, New York City. He was transferred to take charge of sinking the pneumatic caissons under the Manhattan Life Building, New York City.

From 1894 to 1897 Mr. Thomas was engaged as a civil and consulting engineer in Chicago, Illinois, among his works being new piers for the Glasgow Bridge, construction contracts for the Lake Street "L" Railroad in Chicago, and the erection of a railroad bridge on the St. Joseph River, near Benton Harbor, Michigan.

From October, 1897, to October, 1898, he was called in to finish building ten piers and several abutments previously begun for the New York and Ottawa Railroad Bridge, at Cornwall, Ontario. At the conclusion of that contract he became associated with the Engineering Contract Company, and for them executed the following works: In the winter of 1898-99 the extensive repairs to the large dam of the Royal Electric Company at Chambly Canton, Quebec; and in the working seasons of 1899 and 1900 the construction of the piers for the Midland Railroad Bridge at South Maitland, Nova Scotia. This was a very difficult work, as it required the placing and sinking of five pneumatic caissons in a river having a rise and fall of tide of from 24 to 32 ft., and a current of from 8 to 12 miles per hour. Operations had to be carried on between tides, owing to the fact that the incoming and outgoing water would carry out all unsecured work.

In the spring of 1900 he started the shafts for the Derby Lead Company, at Elvins, Missouri. In the summer of 1901 Mr. Thomas took charge of the construction of the foundations for the Marine Engineering Building at the Naval Academy, Annapolis, Maryland, for the Snare and Triest Company, of New York City. In the spring of 1902 he resigned, and accepted the position of engineer for Noel and Thomas, of Baltimore, Maryland, contractors for the construction of the Cadet Quarters at the Naval Academy.

In the fall of 1902 Mr. Thomas became Supervising Engineer for the Interborough Rapid Transit Company, of New York City, in the construction of its main power-house at 59th Street and North River, a work of immense magnitude, and costing about $7 000 000.

In his paper,* "Deep and Difficult Bridge and Building Foundations," Mr. Thomas describes many details of the difficult foundation work done by him, and some of his experience with the freezing process for sinking shafts through water-bearing soil is described in his discussion of the paper† "Freezing as an Aid to Excavation in Unstable Material," by James H. Brace, Assoc. M. Am. Soc. C. E.

On March 1st, 1906, he became supervising engineer for Mr. Francis H. Kimball, Architect, on the Brunswick Building, New York City, and in June he was transferred by Mr. Kimball to supervise the con-

* *Journal*, Western Society of Engineers, Vol. I, 1896, p. 437.

† *Transactions*, Am. Soc. C. E., Vol. LII, 1904, p. 437.

struction of the City Investing Building, a 34-story structure on Broadway and Cortlandt Street, New York City. He was engaged on this work at the time of his death. Concerning him, Mr. Kimball writes:

"His experience and skill in his profession were of great value, and I regret that his mantle cannot fall on someone who would be able to carry on his work. He has taught many what they know, but I know of no one so clear headed in time of need. Mr. Thomas leaves to his family a splendid reputation for skill in his calling, and a rugged, honest nature which stood out in bold relief among his fellows. No one knew him who did not respect him, and his loss is deplored by all."

Mr. Thomas always commanded the confidence of those he served and of those who served under him. The former trusted him because of his ability, because what he undertook he carried through successfully; while the latter, who worked with him, worked for him. He had their confidence because he was a leader who could not only direct but could practically carry out his plans. Men were always willing to follow him. In his hazardous undertakings he never ordered a man to go where he himself would not personally lead. He was a man of rare executive ability, and his advice and services were always in demand.

Mr. Thomas was an active worker in church affairs, and at the time of his death was a steward and class leader in St. Andrew's Methodist Episcopal Church, New York City. He was also a Knight Templar. No man, however humble, who ever came to him for help or counsel went away empty-handed. He had a good word and a smile for all. His men not only respected but loved him.

In response to the many requests made to have him write the story of his life, his invariable reply was, "Oh, I'm too busy living; wait till I retire." He has written his life on the hearts of his fellow men, and on the towering tablets of his great and successful works.

He was elected a Member of the American Society of Civil Engineers on October 5th, 1898, and was for many years a member of the Western Society of Engineers.

William P. Morse

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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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NICKEL STEEL FOR BRIDGES.*

By J. A. L. Waddell, M. Am. Soc. C. E.

In November, 1903, the writer began a series of experiments, upon the comparative values of nickel steel and carbon steel for bridge building, for the purpose of preparing from them an exhaustive economic study of the subject of "Nickel Steel for Bridges." These experiments extended over three years.

The first step was to collect and read all the literature upon nickel steel bearing upon the subject, and, as this was rather meager, the task was soon completed. Although much was learned in this way concerning nickel steel in general, little was ascertained about the composition of the alloy most suitable for bridge building.

At the outset, the writer retained the Osborn Engineering Company, of Cleveland, Ohio, to conduct the experiments, under his immediate direction and supervision; and F. C. Osborn, M. Am. Soc. C. E., the President of that company, at first gave his personal attention to the work, but, as soon as it was laid out, George C. Saunders, Assoc. M. Am. Soc. C. E., was placed in charge for the company.

* This paper, as originally prepared, was much longer and more elaborate than as herein given; but, for the sake of economy of space, its dimensions have been reduced in every possible way that did not injure its continuity or militate materially against its ultimate object. The original paper with its six appendices is to be found in the Library of the Society, where those who desire any of the detailed information it contains and this paper does not, may refer to it. As it gives every step in the development of the investigation, and shows how the writer's ideas and intentions were modified from time to time as the inquiry proceeded, it may prove of interest to those who are contemplating making important engineering investigations.

This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

In order to ascertain what was known of nickel steel suitable for bridge building which had not been recorded in print, the writer, in company with Mr. Osborn, made trips to Pittsburg and Pencoyd to consult with various officers and employees of the Carnegie Steel Company and the American Bridge Company. The result of these visits was very satisfactory; for much valuable information was thus obtained, and some material from two old melts of nickel steel was secured for the preliminary testing.

At the initiation of the work, a lay-out of tests covering the entire proposed investigation was prepared, and this was followed quite closely; nevertheless, as the testing proceeded, it appeared advisable to drop certain tests and to add others. One large group had to be abandoned on account of expense, as it involved the provision of another large melt of nickel steel; but from a number of tests on some other nickel steels the necessary deductions could be made.

The initial lay-out of the work involved the following:

1st.—The determination of the proportions of nickel and carbon that will provide a steel of the highest ultimate strength and elastic limit, consistent with perfectly satisfactory manipulation of the metal in the shops, and with absolute safety in the operation of structures built of it;

2d.—The making of a complete series of tests of the nickel steel or steels adopted as standard, and a corresponding series of tests of the ordinary carbon steel used in the manufacture of bridges, so as to determine the values of all intensities of working stresses for nickel steel in bridges;

3d.—The preparation of a thorough and minutely detailed set of specifications governing the designing of railroad and highway bridges in nickel steel;

4th.—The preparation of diagrams of weights of metal per linear foot of span, for all ordinary types of both single-track and double-track railway bridges, and for all spans, from that of the shortest plate girder up to that of the longest practicable cantilever, all designed for modern live loads and according to standard specifications, these diagrams to establish the weight curves for structures entirely of carbon steel, those entirely of nickel steel, and those of mixed nickel and carbon steels, in which the latter material is used for all parts where there would be no advantage in having the stronger metal;

5th.—The preparation of a set of diagrams giving in comparison the total actual costs per linear foot of span for metal erected in nickel-steel and carbon-steel structures, and in mixed-steel and carbon-steel structures, for all the bridges of which the weights were diagrammed, and for all possible pound prices for carbon steel erected (varying by ½ cent), and for all possible variations in pound prices of manufactured nickel steel and manufactured carbon steel delivered at the bridge site. These diagrams would enable anyone to perceive at a glance the economics of nickel steel for all standard types of railroad bridges and for all possible conditions of the metal market; and, moreover, they would be good for all time, provided the characteristics of the two metals remained constant.

The initial step in making the investigation was, of course, to determine the best percentage of nickel to use. At first thought, this appeared to be purely a commercial question, the idea being that the greater the percentage of nickel the greater the strength of the alloy, but, at the same time, the greater the cost per pound, and that, consequently, there must be some one percentage of nickel which is more economical than any other. This is true enough, but the enquiries made at Pittsburg indicated that when the percentage of nickel exceeds a certain amount (then not determined definitely) the material becomes too refractory for shop manipulation or too expensive in manufacturing.

At the outset the writer had expected that it would be necessary to make a number of small special melts of nickel steel containing varying proportions of nickel, carbon, and possibly manganese, in order to settle the question of the best composition for the alloy; but the enquiries referred to and the preliminary tests on the specimens of nickel steel secured enabled him to determine this point without that necessity. Curiously enough, the superior limit of nickel for workability in bridge shops varies but little, if any, from the economic amount based on present prices of steel and nickel. For plates and shapes, which form the principal part of bridge superstructures, any materially greater percentage of nickel than 3½ renders the alloy too refractory for the various shop manipulations to which it would be subjected in manufacturing it into bridges, although superstructures built of steel still higher in nickel would be perfectly safe and satisfactory for operation.

The writer learned also, both by enquiries and by the preliminary tests, the best percentages of carbon to adopt for nickel steels of the various kinds required in bridge building; and he is of the opinion that any elaborate investigation of this question that might be made would not modify his conclusions materially. These best percentages of carbon will be indicated later, in connection with the discussion of the tests. For the present it will suffice to make the general statement that the greater the percentage of nickel in the alloy the greater the permissible amount of carbon, as far as safety against brittleness is concerned; and that, as the addition of carbon to the steel costs nothing and adds materially to both the ultimate strength and the elastic limit of the metal, it is economical to use as much of it as a proper consideration of workability in the shop and safety in operation of structure will permit.

After making the enquiries at Pittsburg and Pencoyd, the writer, in consultation with Mr. Osborn, adopted the following maximum percentages of impurities in the nickel steel recommended for bridge work:

Phosphorus	0.03%
Sulphur	0.04%
Silicon	0.04%

At the same time it was decided to leave acid open-hearth steel out of consideration, and to assume that structural nickel steel will be manufactured exclusively by the basic open-hearth process.

As for the proper percentage of manganese, it was decided at first that this should be limited to about 0.6%; but, later, it was found advisable to increase this to 0.75% for plate-and-shape steel, and to 0.85% for eye-bar steel. Manganese adds to the strength of the alloy and costs comparatively little, but its use in excess renders the metal unduly hard.

The following is a list of the comparative tests of nickel steel and carbon steel which were made in the investigation:

A.—Specimen tests for tension, showing the elastic limit, ultimate strength, elongation, and reduction of area for the several kinds of steel required,

B.—Bending tests on specimens, both plain and nicked, to determine the angle of bend and the character of fracture,

C.—Bending on pins, to ascertain the extreme fiber stress for both elastic limit and ultimate strength,

D.—Bearing on pins and rivets, to determine the proper intensities of working stresses for bearing,

E.—Shear on rivets, to find the correct intensity of working stress for shearing,

F.—Tests for torsion, to ascertain the safe limit of extreme fiber stress on shafts of operating machinery,

G.—Impact tests, to learn the relative resiliences of nickel steel and carbon steel,

H.—Combined impact and tension tests, to show the comparative resistances of the two metals for this combination of conditions,

I.—Tests of full-sized eye-bars,

J.—Tests of full-sized columns, both short and long, to aid in the preparation of proper formulas for nickel-steel struts,

K.—Hammering-flat tests,

L.—Drifting tests,

M.—Close-punching tests,

N.—Planing tests,

O.—Drilling tests,

P.—Chipping tests,

Q.—Filing tests,

R.—Specific gravity tests,

S.—Coefficient of elasticity tests,

T.—Corrosion tests, to determine the comparative resistances of the two metals to the attacks of acid, damp salt, smoke, and wet cinders.

Analyses of the metal from the two old melts of nickel steel obtained for testing, and the medium-carbon steel used for comparison, gave the results shown in Table 1.

TABLE 1.—Results of Analyses of Steels.

Character of alloy.	Percentages of Ingredients.				
	Sulphur.	Phosphorus.	Manganese.	Carbon.	Nickel.
Low-nickel steel.....	0.015	0.011	0.65	0.39	3.21
High-nickel steel.....	0.014	0.009	0.67	0.463	4.25
Medium-carbon steel.....	0.021	0.011	0.46	0.275	....

The percentages in Table 1 were obtained from drillings; but analyses of samples taken from the ladle gave, for the low-nickel steel, manganese, 0.68, carbon, 0.35, and nickel, 3.20; and for the high-nickel steel, manganese, 0.65, carbon, 0.42, and nickel, 4.22.

The following is a condensed record of all the preliminary tests, which are given in full in Appendix C.*

Tensile Tests.

An average of eleven tensile tests of the low-nickel steel gave 61 300 lb. per sq. in. for the elastic limit and 99 300 lb. per sq. in. for the ultimate strength. An average of fourteen tests of the high-nickel steel gave 71 600 lb. per sq. in. for the elastic limit and 114 000 lb. per sq. in. for the ultimate strength.

Bending Tests.

The bends were made on unannealed specimens, 2 in. wide and 10.5 in. long, with planed edges. The warm bends were made at ordinary shop temperatures, ranging from 55 to 70°, and the cold bends after the specimens had been immersed for 2 hours in a mixture of salt and ice. Four warm and four cold bends were made on each thickness of plates and angles of each of the three kinds of steel. The effect of the low temperature was quite marked. It seemed to be greater in the high- than in the low-nickel steel; but the tests were too few in number to warrant stating this conclusively.

All the low-nickel steel at the ordinary shop temperature was bent 180° about two diameters; the high-nickel steel was bent 120° about three diameters. The cracks developed much more suddenly in the high-nickel steel and in the cold bends than in the low-nickel steel and in the warm bends, and were greater in magnitude. The bends of the low-nickel steel specimens were better than those of the high-nickel steel, but were not as good as those of the carbon-steel specimens.

Impact Tests.

There were four different series of impact tests. The first was made on specimens 21 in. long, $\frac{3}{4}$ in. wide and 2 in. deep, reduced at mid-length by two notches, each $\frac{1}{2}$ in. wide and $\frac{1}{2}$ in. deep, leaving an

* This appendix forms a part of the original paper, which may be found in the Society's Library. It consists of an itemized report of all the preliminary tests by Mr. Saunders.

effective depth of only 1 in. Five pieces of each kind of steel were broken, the specimens being reversed after each blow.

The apparatus, Fig. 1, was a rather crude affair, and was operated with much labor. The weight was a 50-lb. box, hoisted by a cord running over a small drum. It was raised and lowered by a worm gear turned by a small crank. Additional weights, in units of 50 lb., were added by placing castings on top of the box. The weight was released by disengaging a catch.

The test pieces were supported on rounds, 12 in. between centers, and the load was transmitted through a tongue rounded at the bottom so as to bring the effect of the blow midway between the supports.

Contrary to what had been gathered from the perusal of a number of papers on nickel steel, it was found by these experiments that the resilience of the nickel-steel alloys was considerably less than that of the carbon steel. All three steels were submitted to the same impact tests, the weight at first being 100 lb. and the drop increasing gradually from 6 to 24 in.; then, after twelve blows, the weight was increased to 150 lb. Calling the resulting average resilience of the carbon steel 100, that for low-nickel steel was 87 and that for high-nickel steel was 73.

This result might have been predicted, for turning over the specimens after each blow caused the total amount of abuse given to the metal to be a measure of its toughness and not of its resilience.

The second series of impact tests was similar to the first, except that the specimens were not reversed. The result gave the following comparison of resiliences:

Carbon steel	100
Low-nickel steel	117
High-nickel steel	109

Because of the contradictory results of the two series of impact tests, it was decided to make a third, in order to determine the relative resistances of the three steels to a sudden blow.

To make the tests, the apparatus shown in Fig. 2 was constructed. The hammer was forged round, with an eye-bolt in the top, and weighed 205 lb. It was raised by a traveling overhead crane. The specimens were similar to those used in the first series of tests, except that their length was reduced to 18 in., and there were ten of each of the three kinds of steel.

No piece was struck more than one blow, and the height of drop first selected was such as would positively break the specimen. With the succeeding diminishing heights, an effort was made to approximate to that which would just break the piece.

The comparative resiliences found by this series of tests were:

Carbon steel	100
Low-nickel steel	78
High-nickel steel	71

Thinking that the unsatisfactory showing of nickel steel under impact, as compared with carbon steel, and the diametrically divergent results of these impact tests, as compared with the results recorded by other experimenters, might be due to the notchings of the specimens, a fourth series, similar to the third, was made, but using plain bars of a section corresponding to that of the notched portion of the specimens previously tested, or $\frac{3}{4}$ in. in width and 1 in. in depth. It proved necessary to use a 500-lb. hammer, as the 200-lb. hammer used previously was entirely insufficient to break the specimens or to bend them materially. As it was, the tests were imperfect and inconclusive, for the high-nickel steel specimens broke under a drop of about 9.5 ft., those of low-nickel steel bent through an angle of from 120 to 130° under a drop of 12 or 13 ft. (only one piece breaking, and the others showing very slight indications of fracture), and those of carbon-steel bent under a drop of about 13 ft. without showing any signs of fracture. As the limits of the apparatus were reached, no attempt was made to carry the experiments any further.

Although the result was imperfect, the test is quite satisfactory in that it shows clearly the great amount of abuse that low-nickel steel will stand, and that, in this particular, it is quite suitable for bridge building; although, of course, it is not as tough and ductile as carbon steel. Had the metal been at all brittle, the specimens would have been shattered instantly by so large a weight falling from so great a height, instead of first bending through such large angles.

Up to the present, the general consensus of opinion has been that nickel steel has a higher resilience than carbon steel, and it was a matter of surprise to the writer that three of his four series of impact experiments indicated the contrary. On this account a thorough discussion of this feature of the paper is desirable. It may be concluded

that at present the comparative resiliences of nickel steel and carbon steel are still an open question; and it is hoped that the discussion will throw sufficient light upon the matter to settle finally the existing uncertainty.

It must not be forgotten that, in comparing the resiliences of nickel and carbon steels, if the strengths of the two metals be equal, or if their carbon contents be equal, nickel steel will give better results than carbon steel; and it is only when comparing nickel steel high in carbon with the ordinary low-carbon steel used in bridgework that the showing is ever unfavorable to the alloyed metal.

Combined Impact and Tension Tests.

The writer made an attempt to ascertain the effect of dropping a weight upon an eye-bar under tension, but the test was a failure. The weight (200 lb.) and the drop (4 ft.) were both too small. Although the bar was finally broken, it was impracticable to determine whether the ultimate strength was affected by the impact, as the piece developed a pretty high resistance for medium-carbon steel.

To obtain any results of value, it would have been necessary to set up a small pile-driver over the bar and drop a much larger weight from a greater height and with far greater frequency. After this experience it was concluded that "the game was not worth the candle," consequently the intention of carrying this test any further was abandoned.

Later, the writer retained W. K. Hatt, Assoc. M. Am. Soc. C. E., of Purdue University, to make, among other tests, some experiments on combined tension and impact, and combined bending and impact. His full report is given in Appendix E.* The following extracts from it will show briefly his findings:

"The impact tension tests were made on the machine in the Purdue Laboratory used by the writer in previous tests, and described in the *Proceedings* of the American Society for Testing Materials, Vol. 4, 1904. The hammer used weighed 810 lb. This was allowed to drop a distance, h, in such a way that the energy of the blow was absorbed by the test piece. If the piece broke under the first blow, a record, taken on a revolving drum, made it possible to compute the energy left in the hammer after rupture. In nearly every case the piece was ruptured by one blow.

* On file in the Library of the Society, but not reproduced in this paper.

. "The impact flexure tests were made upon an electrically-operated impact machine. The hammer used weighed 55 lb. The 3 600-lb. base was supported by a 6-ft. concrete foundation resting upon undisturbed gravel. The bars were tested on a span of 18 in., with the machined faces on the top and bottom. The loads were applied by allowing the hammer to drop upon the center of the beam from varying heights, without turning the bar over between blows. The corresponding deflections were indicated by a recording pencil on the slowly revolving drum. The routine of each test was as follows:

"With the hammer just touching the bar, a zero line was marked on the drum. The hammer was then allowed to fall from a height of $\frac{1}{4}$ in. on the bar, the deflection being marked on the slowly revolving drum. These operations were repeated, increasing the height of drop $\frac{1}{4}$ in. each time, until the elastic limit was well passed. When the elastic limit was reached, the material showed signs of set. This was marked on the drum by the recording pencil by turning the drum slightly after the vibrations of the bar had ceased. A typical drum record is shown on Fig. 3. None of the specimens, except the nicked bars, were broken in flexure.

"Two of the nicked specimens, * * * one of carbon steel and one of nickel steel, were tested in a different manner from that outlined above. The hammer was allowed to fall from a height of 18 in., then from a height of 24 in., and then from a height sufficient to cause rupture. The drum records for these tests were similar to those obtained in the impact tension tests."

Professor Hatt's results of impact tests are given in Table 2, which forms a part of his report.

TABLE 2.—Results of Impact Tests.

	Carbon.				Nickel (3.5%).			
	No.	Average.	Maximum.	Minimum.	No.	Average.	Maximum.	Minimum.
Impact Tension.								
Elongation	5	31.5	32.0	31.0	4	16.5	19	13
Contraction	5	58.9	60.8	57.0	4	49.7	54	45
Rupture-work	5	1 736	1 910	1 540	3	2 832	3 207	2 726
Impact Flexure. Plain bar.								
Deflection at blow from 3 in	8	0.18 in.	0.21 in.	0.16 in.	5	0.09 in.	0.11 in.	0.09 in.
Deflection at elastic limit	8	0.21 "	0.23 "	0.16 "	5	0.26 "	0.30 "	0.22 "
Height of drop at elastic limit	8	3.70 "	3.50 "	3.00 "	5	8.10 "	9.00 "	7.25 "
Impact Flexure. Nicked bar.								
Deflection at blow from 3 in	3	0.20 in.	0.22 in.	0.17 in.	3	0.12 in.	0.14 in.	0.12 in.
Deflection at elastic limit	3	0.21 "	0.25 "	0.17 "	3	0.23 "	0.24 "	0.21 "
Deflection at rupture	3	1.23 "	1.40 "	0.9 "	3	0.62 "	0.70 "	0.57 "
Height of drop at elastic limit	3	3.30 "	3.57 "	3.25 "	3	6.10 "	6.25 "	6.00 "
Height of drop at rupture	3	11.10 "	12.75 "	9.50 "	3	15.60 "	17.00 "	14.5 "

Table 2 shows that, in the impact tension tests, while the elongation of carbon steel was nearly twice as great as that of nickel steel and the contraction was more than 20% greater, nickel steel had a resilience 60% higher than carbon steel—a result almost entirely at variance with the writer's findings.

Table 2 also shows, in the impact flexure on plain bars, an increase in resilience of nickel steel over carbon steel equal to 120% at the elastic limit, and in the case of nicked bars an increase of 85% at the elastic limit and 40% at the point of rupture.

Hammering-Flat Tests.

The hammering-flat tests were quite conclusive, and proved that both the nickel steels compare very favorably with carbon steel in their ability to permit of being hammered flat.

Drifting Tests.

The drifting tests, as anticipated, show that nickel steels will not stand quite as much drifting as medium-carbon steel, nevertheless, both grades would comply with the drifting requirements for high-carbon steel given in standard bridge specifications.

Close-Punching Tests.

The experiments show that both the nickel steels withstood close-punching quite as well as the carbon steel, in fact, in one respect better, because the flow of metal was markedly less in the stronger steels, and the holes were smoother and more regular.

Shop-Tooling Tests.

The shop-tooling tests included planing, drilling, chipping, and filing. They were quite satisfactory, and gave the best comparative results of all the tests made. They showed conclusively that high-nickel steel is unfit for bridge building (except for eye-bars, pins, and rollers), not because of any inherent defect in the alloy, but because of its severity on the tools. To manipulate such metal in the shops, all machinery and tools would have to be built upon a more substantial basis than for carbon-steel work, and a stronger metal for punches and drills would have to be adopted.

As for low-nickel steel, the tests showed that this alloy is all right for shop manipulation, but the time required to do a certain amount of work upon it is greater than that required to do the corresponding work upon carbon steel.

Corrosion Tests.

The corrosion tests included immersion in a weak solution of sulphuric acid, in damp salt, in locomotive fumes, and in wet cinders. In order to avoid the possibility of accident, the writer decided to have these tests made in duplicate—in his own office, at Kansas City, and in the office of the Osborn Engineering Company, at Cleveland. Owing to failure to ensure in advance that the methods of testing would be exactly alike in detail, the results are not truly comparable.

All the plates tested were originally 6 by 6 by $\frac{1}{2}$ in., and weighed about 5 lb. each. Three plates (one for each kind of steel) were used for each test, or twelve in all for each set of experiments.

Acid Test.—Mr. Osborn used a 1% solution of sulphuric acid, with a new bath every two weeks, and he did not clean the specimens; while the writer used a 2% solution, changed the bath every week, and scraped the specimens each time the bath was changed. The results of these tests are recorded on Fig. 4. From the writer's record it will be seen that in 160 days 94% of the high-nickel steel was dissolved, thus ending the experiment, and that in the same time the low-nickel steel lost 89% and the carbon-steel 52 per cent. Mr. Osborn found in 890 days the following losses of weight: 13.2, 12.9, and 11.7%, respectively.

Both investigators were surprised by the results of this test, because they had been told by one of the Carnegie Steel Company's superintendents that the pickle test would show up very favorably for nickel steel, as he knew by having made the experiment.

This test, however, proves nothing against the use of nickel steel for bridges, because those structures are never exposed to such a condition or to any condition even approximating to it in severity, and because the other three corrosion tests, which agree in character more closely with the actual exposures of bridge metal, indicate a decided superiority of nickel steel over carbon steel.

Salt Tests.—As shown by Fig. 5, the writer's salt test extended over 522 days, at the end of which time the carbon steel had lost

2.65% of its weight, the low-nickel steel 0.72%, and the high-nickel steel 0.63%, showing that nickel steel resists the attacks of salt four times as well as carbon steel. Whether the two steels when immersed in sea water would offer the same comparative resistances to corrosion is hard to say, but it seems probable that they would. If so, this would be a strong point in favor of using nickel steel for ocean piers and for bridges located near salt water.

Mr. Osborn's results for the salt test were: In 840 days the carbon steel had lost 3.3%, the low-nickel steel 3.1%, and the high-nickel steel 3.2 per cent. The curves at 880 days cross each other, but this is probably due to some error in the records, since for 800 days the three curves are substantially parallel.

The great variation in the two sets of experiments must be due to the fact that the writer's plates were kept continuously in the solution and Mr. Osborn's were not. While this difference in treatment might account for the variation in the total amount of corrosion for any one kind of plate, it does not explain why the comparative resistances of the three kinds of steel should differ so greatly in the two sets of experiments.

Smoke Tests.—As shown by Fig. 6, the smoke test was continued by the writer for 250 days, when the plates were lost by the renewal of the floor system of the bridge beneath which they were suspended. His test at the end of that time showed a loss of 6.25% of its weight by the carbon steel, 4.5% by the low-nickel steel, and 4.4% by the high-nickel steel, indicating that nickel steel resists locomotive fumes about 40% better than carbon steel.

Mr. Osborn found the following results: In 310 days the carbon steel lost 0.18%, the low-nickel steel 0.12%, and the high-nickel steel 0.04 per cent.

The great difference in the results of the two tests was certainly caused by Mr. Osborn's failure to clean the plates from time to time, for the coat of soot deposited on the metal tended to preserve it from further attacks of the fumes.

Cinder Tests.—As shown by Fig. 7, the cinder test was continued by the writer for 520 days, at the end of which time the carbon steel had lost 3.45% of its weight, the low-nickel steel 2.5%, and the high-nickel steel 2.15 per cent. This is quite a favorable showing for the nickel steel.

Mr. Osborn's cinder tests gave the following results: In 890 days the carbon steel had lost 16.7%, the low-nickel steel 14.5%, and the high-nickel steel 14.1 per cent. Up to 740 days, however, the high-nickel steel corroded a little more than the low-nickel steel. The greater amounts of corrosion in the Osborn test must have been due to the nature of the cinders used.

From the various corrosion tests made by both experimenters, it may be concluded that nickel steel resists decidedly better than carbon steel the attacks of all substances which ordinarily tend to injure the metal-work of bridges.

Rivet Steel.

While the various preliminary tests were being made, a thorough investigation of the qualities of a suitable nickel steel for rivets was carried on, using specimens of old melts, in order to avoid the expense of making a special melt or melts for the purpose. While none of the five or six melts tested proved exactly suitable for rivets, enough was learned by the experiments to determine closely the proper proportions of the various constituents of rivet nickel steel. All the specimens tested were good enough for the purpose, except in one important particular, namely, the rivets were too difficult to cut out, the reason being that the carbon contents were too high.

In the writer's opinion, ideal rivet nickel steel should have about the following composition:

Nickel	3.5 per cent.
Carbon	0.12 to 0.18 per cent.
Manganese	0.55 to 0.65 per cent.
Phosphorus	0.03%, maximum.
Sulphur	0.04%, maximum.
Silicon	0.04%, maximum.

It is possible that a small economy might be effected by lowering the percentage of nickel and increasing that of carbon, and in time this may be done; but at present it does not seem worth while to make the nickel content less than that in plate-and-shape steel. The writer is of the opinion that by experimenting it could probably be shown that a rivet steel containing from 2 to 2.5% of nickel and from 20 to 25 points of carbon would answer every purpose; but this is only a surmise on his part. It is possible, though, that such a high percentage

of carbon would cause the rivets to burn too easily, but the writer's experiments gave no such indication.

In addition to the ordinary tests for tension and bending, a number of special tests on rivet nickel steel were made, among others, the following:

A.—Driving two rows of nickel-steel rivets and two of carbon-steel rivets so as to connect four thicknesses of ½ by 12-in. by 3 ft. 6-in. plates, reaming one row of holes for each kind, and noting how the nickel-steel rivets drove, as compared with the carbon-steel rivets:

The result was that no difference whatsoever could be noticed in the difficulty of driving or the excellence of the rivets, and this was found to be true throughout the entire series of experiments on nickel steel and carbon steel.

B.—Cutting off by hand the heads of most of the rivets mentioned in *A*, and driving them out of their holes, so as to obtain a proper comparison by noting the times required:

The results of this test were very erratic, but, in general, it was found that the higher in carbon the nickel steel the longer it took to cut off the heads. When a pneumatic chipper was used, it sometimes required twice as much time for nickel steel as for carbon steel.

In backing rivets out of reamed holes, no difference was noted in the resistances of the two kinds of rivets; but, in punched holes, the nickel-steel rivets came out in three-quarters of the time required for backing out the carbon-steel rivets. This result is due to the fact that the punched holes in nickel steel are much smoother than those in carbon steel. In this test (and in others also) it was learned that nickel-steel rivets should be used in nickel-steel plates and carbon-steel rivets in carbon-steel plates; for, when cutting out nickel-steel rivets from carbon-steel plates, the material around the rivet holes was injured because of the greater hardness of the rivet metal.

C.—Punching four ½ by 6 by 15-in. plates with two rows of badly-matched holes (but making the matching in the two rows correspond), assembling the plates, reaming the holes, and filling one row with nickel-steel rivets and the other with carbon-steel rivets, then planing off the sides of the plates up to the medial planes of the rivets, so as to show how well the holes were filled:

It resulted that there was but little difference in the flow in the two kinds of steel, what little there was being in favor of carbon steel.

D.—Flattening the ends of $\frac{7}{8}$-in. rivets, both cold and hot, and noting how the metal stands the abuse:

The cold nickel-steel rivets flattened to $\frac{7}{16}$ in. and then split, while the ends of the carbon-steel rivets flattened to $\frac{3}{16}$ in. before splitting. Undoubtedly, there would have been a better showing with nickel steel of ideal composition for rivets. The hot ends of both steels flattened out very thin without cracking.

E.—Flattening rivets, both cold and hot, end on:

In this test the cold carbon-steel rivets compressed 67% and the cold nickel-steel rivets 45 per cent. The latter required a much greater number of blows and received more damage. In the case of the hot rivets, both specimens flattened to a thickness of $\frac{3}{16}$ in. and to a diameter of $3\frac{1}{2}$ in. without showing signs of cracking.

F.—Testing rivets in double shear:

The result of the tests of rivets in double shear showed that the nickel steel was about 40% stronger than the carbon steel, the ultimate strength of rivet nickel steel in shear being about 59 000 lb. per sq. in., or about 75% of the tensile strength.

G.—Testing riveted joints:

The result of the test on riveted joints was to obtain an ultimate shear of 68 000 lb. per sq. in. on the nickel-steel rivets and 46 500 lb. per sq. in. on the carbon-steel rivets. The ratio of these two figures is about 1.46, which is a little greater than the ratio found for the shears on single rivets.

All things considered, the writer favors adopting rivet diameters for nickel-steelwork $\frac{1}{8}$ in. greater than those customary for carbon-steelwork. There are three good reasons for this, namely:

First.—The increase of strength of nickel-steel rivets over carbon-steel rivets is not commensurate with the increase of strength in the plate-and-shape metal. This necessitates either a greater proportionate number of rivets or a greater rivet diameter in nickel-steelwork.

Second.—The larger rivets keep hotter, and consequently fill the holes better.

Third.—The strength of the punches is increased about one-third, while the work of punching each hole is increased only about 13 per cent. This reduces the danger from broken punches, and makes the work in punching nickel steel probably not more hazardous than that in punching carbon steel.

Eye-Bars of High-Nickel Steel.

In order to determine approximately the kind of material for eye-bars the high-nickel steel would make, there were cut from the $\frac{3}{4}$-in. plates two strips, 5 in. wide, and these were forged into eye-bars, annealed, and tested.

The elastic limit of the first bar broken was lost. Each bar broke near the neck, at a point where the temperature stresses are the greatest, and this, in connection with the low percentage of elongation in the body of the bar, would seem to indicate insufficient annealing. The breaking of both bars in the same place, however, may have been simply a coincidence. The fractures were 100% silky, and the reductions of area were excellent.

The results of the two tests were as follows:

Number of test.	Elastic limit.	Ultimate intensity.	Percentage of elongation in 10 ft.	Reduction of area.	Fracture.
Bar 1.....	Lost.....	105 900	7.4	46.9%	Silky, $\frac{1}{2}$ cup.
Bar 2.....	Uncertain	102 300	6.8	45.8%	Silky, $\frac{1}{2}$ cup.

The unannealed specimen tests on the same plate gave an elastic limit of 71 100, an ultimate strength of 112 900, an elongation in 8 in. of 17.7%, and a reduction of area of 44.4 per cent.

These tests are valuable in that they prove that satisfactory eye-bars can be manufactured from steel containing $4\frac{1}{4}$% of nickel and about 45 points of carbon.

Special Melts of Nickel Steel.

Two special 40-ton melts of plate-and-shape steel were made for the writer by the Carnegie Steel Company, under the supervision of Albert Ladd Colby, M. Am. Soc. C. E., to whose care and attention is due the fact that the metal obtained was greatly superior in uniformity to the low-nickel steel first tested. The composition of these special melts was determined by a number of independent chemists, with rather variable results, the general averages being as follows:

Nickel 3.65%, carbon 0.39%, manganese 0.77%, sulphur 0.025%, phosphorus 0.01%, and silicon 0.05 per cent.

The metal from these melts was rolled into plates and angles, some of which were used by the writer for testing. The remainder

was distributed among the principal bridge shops of the country to undergo various manipulations, in order to let the manufacturers judge for themselves of the ease or difficulty of manufacturing it into bridges. This distribution was done liberally, even lavishly, and it is hoped that those who received the metal have submitted it to many practical shop tests, and that they will give the results thereof to the engineering profession in the discussion of this paper.

The following is a list of the companies to whom material was thus sent:

Dominion Bridge Co., Ltd., Montreal, Que.;
Riter-Conley Mfg. Co., Pittsburg, Pa.;
The Canadian Bridge Co., Walkerville, Ont.;
The Hamilton Bridge Works Co., Ltd., Hamilton, Ont.;
Milliken Bros., New York City;
The King Bridge Co., Cleveland, Ohio;
McClintic-Marshall Construction Co., Pittsburg, Pa.;
Pennsylvania Steel Co., Steelton, Pa.;
Fore River Ship and Engine Co., Quincy, Mass.;
Newport News Shipbuilding and Dry Dock Co., Newport News, Va.;
William Cramp and Sons Ship and Engine Building Co., Philadelphia, Pa.

It was decided to postpone making the special melt of eye-bar steel until after the plate-and-shape steel had been tested. This was a wise precaution, because the testing of the latter might have caused some modification of the composition of the eye-bar steel; but the result, unfortunately, was disastrous, as the eye-bar steel was never manufactured. This failure reduces somewhat the value of this research, as compared with what was outlined when the experiments were first contemplated. The eye-bars for testing were manufactured from the plate-and-shape steel; and, although the results were very satisfactory, the writer is convinced that much stronger eye-bars can be readily manufactured by using steel higher in nickel, carbon, and manganese, and that they will be suitable in every way for use in bridges.

The special plate-and-shape nickel steel was tested very thoroughly and carefully, both in specimens and full-sized bridge members. Except in the case of eye-bars, for each test of nickel steel there was

made a corresponding test of medium-carbon bridge steel. The results are recorded at length in Mr. Saunders' report, which is given in the original paper,* and is reproduced in a slightly condensed form in Appendix A of this paper.

From it the writer has prepared the following summary and deductions:

Tensile Tests.

The average elastic limit is about 62 200 lb. per sq. in., and the minimum may safely be taken at 60 000 lb. for specimens cut from the edge; the average ultimate strength is about 107 300 lb. per sq. in., and the minimum may safely be taken at 105 000 lb. for edge specimens. For specimens taken from the edges of plates, the ultimate strength is about 3 000 lb. per sq. in. higher, and the elastic limit about 2 000 lb. per sq. in. higher than for specimens taken from the interior. This ruling does not appear to apply to eye-bar flats, for their interior metal is just as strong as, if not stronger than, the metal near the edges.

The elongation increases slightly with the thickness of the piece, and the reduction of area decreases as the speed of the testing machine increases.

The elastic ratio (that is, the ratio of elastic limit to ultimate strength) decreases as the thickness of the metal in plates or shapes increases; but the conditions affecting the testing of eye-bar metal were such as to prevent any reliable deduction being made on this point in relation to flats. This elastic ratio, in general, varies from 0.55 to 0.60, but occasionally there are cases that pass slightly beyond these limits.

Comparing the nickel steel and the carbon steel (which was of the kind specified as medium steel in "De Pontibus"), it was found that the average ratio of elastic limits was 1.94 and that of ultimate strengths was 1.79.

The minimum elongation in 8 in. was about 15% for nickel steel and 27% for carbon steel.

The minimum reduction of area was about 41% for nickel steel and about 55% for carbon steel.

It was found that thick plates and angles gave lower elastic limits and ultimate strengths than thin ones. This was already known to be

*On file in the Society's Library.

true for carbon steel, but the difference is so marked in nickel steel that any standard specification for the strength of that metal will have to take into account the thickness of the piece tested.

Difficulty was found in determining the elastic limit, or, more strictly speaking, the yield point; consequently, Mr. Saunders was compelled to make some arbitrary assumptions, as shown by his report.

It was noted during the tests that the nickel-steel angles were slightly inferior in strength to the nickel-steel plates, probably because of the smaller amount of work the angles received in the rolling, and possibly because of the bend. It has been recognized for years in carbon steel that plate specimens give slightly better results than shape specimens.

At the outset, the writer hoped to be able to obtain plate-and-shape steel which would have a minimum elastic limit of 64 000 lb., and that this might possibly be raised to 68 000 lb. by increasing the carbon content to 40 points. The experiments show an inferior limit of 60 000 lb., which might indicate that Mr. Colby's two melts of plate-and-shape steel did not come up to the writer's anticipations; but such is not the case, for, when that metal was tested by the ordinary mill methods, the average elastic limit was 65 600 lb. per sq. in. As the manufacturers' tests on the old melts were made by the usual mill methods, the writer based his judgment of possible results on that style of testing being adopted; but it was soon found that nickel steel should be tested much more slowly than has been customary for carbon steel, and the tests were conducted accordingly. In fact, the usual quick method of testing gives fictitiously great results for both the elastic limit and the ultimate strength of the latter metal. The elastic limit is especially affected. This feature of standard testing will be discussed further, when the subject of eye-bars is treated.

Tensile Tests on Punched, Reamed, and Punched-Riveted Specimens.

The results obtained by this series of tensile tests are as follows:

The punching alone raised the yield point in nickel steel from 2 to 11%, and in carbon steel from 5 to 24%, at the same time lowering the ultimate strength of the nickel steel from 9 to 11% and that of the carbon steel from 6 to 7 per cent.

This shows that the injurious effect of punching is a little more pronounced in nickel steel than in carbon steel.

In the sub-punched-and-reamed specimens, both the nickel steel and the carbon steel regained all their ultimate strength, and in fact a little more. This shows the truth of what has been contended by the writer for a long time concerning reaming, namely, that it is the only method which insures truly first-class shopwork and the full strength of the manipulated metal. It also shows that, while this is true for carbon steel, it is especially so for nickel steel.

The punched-riveted specimen tests indicate that driving the hot rivets into the punched holes increases still further the elastic limit and reduces the ultimate strength, thus furnishing another good reason for insisting upon sub-punching-and-reaming. The injurious effects of driving hot rivets in nickel steel and in carbon steel were about alike for the two metals.

Bending Tests.

The results of the bending tests on plain bars, when the bending is carefully and properly done, indicate that the plate-and-shape steel may be bent 180° around a mandrel having a diameter equal to twice the thickness of the test piece without showing any signs of cracking the metal.

Bending Tests on Punched, Reamed, and Punched-Riveted Specimens.

A study of Fig. 2, Plate CII, of the inspector's report in Appendix A proves very clearly the injurious effects of punching on both nickel steel and carbon steel, for, while the reamed specimens of nickel steel bent on the average about 90°, and those of carbon steel 180°, the punched specimens of nickel steel bent only about 50° and one of carbon steel only 84°; and the punched-riveted specimens of nickel steel averaged only 38°, and one of the carbon-steel specimens bent only 62 degrees.

This test is a strong confirmation of the necessity for sub-punching-and-reaming, and supports well the evidence of the tension tests on punched specimens.

Drifting Tests.

The result of the drifting tests proves that plate-and-shape nickel steel will withstand most of the ordinary drifting requirements for carbon-steel plates, but that, in the new specifications for nickel-steel bridges, the required enlargement of the holes should be limited to 40 per cent.

Close-Punching Test.

This test proves that nickel steel can be punched as closely as carbon steel, and, in fact, that it withstands punching better, in that the edges of the piece are bulged less and that the punchings are smooth and regular while in carbon steel they are rough and irregular. Fig. 1, Plate CIV, might lead one to think that this conclusion is not true, as one specimen shows several broken partitions. These were caused by the nervousness of the operator, who, fearing the breaking of the punch, dodged his head behind a protection at the instant of punching, and thus in several cases got the holes too close together. Nickel steel will stand any specification yet written for the close-punching of carbon steel.

Hammering-Flat Test.

The result of the hammering-flat test shows that nickel steel will stand hammering almost as well as carbon steel.

Shop-Tooling Tests.

The shop-tooling tests were quite elaborate, and covered every important manipulation of metal in a bridge shop. It was found that nickel steel can be sawed with the usual saw and sheared with the usual shears. In punching, there was no special difficulty, but it is probable that more punches would have to be used for nickel steel than for carbon steel. If the diameters of all rivets in nickel steelwork are made $\frac{1}{8}$ in. greater than those for carbon steelwork, as previously mentioned, there should be no greater number of punches broken, because, as before stated, the strength of the punch, which varies as the square of its diameter, would be increased about one-third, while the work of punching would be increased only about 13 per cent. Moreover, it is practicable to improve materially the quality of the metal in the punches.

In reaming, it was found that by the new process, termed "dry reaming," nickel steel can be reamed just as rapidly as carbon steel, but that the tools wear out more frequently. The old-fashioned method of reaming, in which oil or soapsuds are used for the lubricant, is not well adapted to nickel steel.

In drilling, the same remarks apply as in the case of reaming. The ordinary drills fail quickly in nickel steel, but "blue-chip" drills stand

the work very well. The writer's drilling tests were made without a lubricant. Had one been used, it is likely that better results would have been obtained.

In planing nickel steel, it is not practicable to make cuts thicker than $\frac{1}{16}$ in.; while, in carbon steel, cuts of twice that thickness are usual. The effect on the tools is much greater for nickel steel than for carbon steel, and the amount of work that can be done on the former in a given time is much less than can be done on the latter—probably about one-half.

In pneumatic chipping, it takes about 50% more time to make a certain length of cut in nickel steel than in carbon steel, and the amount of metal removed is slightly less for nickel steel.

In hand chipping, a workman will cut in a given time only seven-tenths the length in nickel steel that he will in carbon steel, the thickness of the chips being the same.

Bearing-on-Pins Test.

It was found by the bearing-on-pins test that the plate-and-shape nickel steel is about 66% stronger than the carbon steel. In the case of bearing on rivets the excess of strength amounted to 83 per cent.

Bending-on-Pins Test.

Owing to the difficulty in testing round sections, square ones were substituted, and it was assumed that the comparative resistances for the two metals would be the same for both sections. It was found that nickel steel is about 85% stronger than carbon steel in resistance to bending.

Compression Tests of Struts.

The compression tests of struts were most interesting and important, especially as some engineers of high standing had surmised that there would be no economy in using nickel steel for long columns, because they anticipated that the two kinds of struts would deflect about the same for like total loads. It was found, however, that for short struts the nickel steel was 75% stronger than the carbon steel, and for struts of medium length 47% stronger. While these excesses of strength are not as great as those for tension, they are quite satisfactory.

Coefficient of Elasticity Test.

This portion of the investigation was done with extreme care, three tests being made for each kind of steel, in which the average values of E proved to be 30 312 000 for nickel steel and 29 420 000 for carbon steel. For convenience in calculating, it will be advisable to take the coefficient of elasticity for nickel steel at 30 000 000.

Tests of Eye-Bars and Eye-Bar Material.

Unfortunately, all the eye-bars tested had to be rolled of plate-and-shape steel. Bars 1 and 2 in. thick were used, but no comparison was made with carbon-steel eye-bars of the same sizes. Four 6-in., three 8-in., and one 16-in. bar were tested, and enough specimen tests of the metal were made to afford a proper average. The results of the tests are given in Table 3.

TABLE 3.—Results of Tests of Eye-Bars and Eye-Bar Material.

Six-Inch Bars

	Specimens.			Bars.
Elastic limit	Unannealed	Average	60 600	.....
		Minimum	56 900	.....
	Annealed	Average	54 800	Average 56 400
		Minimum	52 700	Minimum 55 400
Ultimate	Unannealed	Average	103 100	.....
		Minimum	101 300	.....
	Annealed	Average	100 000	Average 101 000
		Minimum	98 700	Minimum 99 000

Eight-Inch Bars.

Elastic limit	Unannealed	Average	58 600	.....
		Minimum	54 700	.....
	Annealed	Average	52 500	Average 51 700
		Minimum	50 700	Minimum 48 300
Ultimate	Unannealed	Average	98 900	.....
		Minimum	93 800	.....
	Annealed	Average	95 200	Average 90 200
		Minimum	92 600	Minimum 88 900

Sixteen-Inch Bars.

Elastic limit	Unannealed	Average	59 100	.....
		Minimum	58 500	.....
	Annealed	Average	55 600	49 400
		Minimum	55 200	
Ultimate	Unannealed	Average	103 600	.....
		Minimum	103 100	.....
	Annealed	Average	103 200	96 900
		Minimum	102 700	

The following deductions have been made from the results given in Table 3:

The average effect of the annealing of specimens is to reduce their elastic limits about 9% and their ultimate strengths about 3 per cent.

The average elastic limits are for:

Unannealed specimens	59 400	lb. per sq. in.
Annealed specimens	54 300	" " " "
Full-sized bars	52 500	" " " "

The average ultimate strengths are for:

Unannealed specimens	101 900	lb. per sq. in.
Annealed specimens	99 500	" " " "
Full-sized bars	96 000	" " " "

The average loss in elastic limit between the unannealed specimens and full-sized bars is about 11%, and the corresponding loss in ultimate strength is about 6 per cent. It is more than likely that, in the future manufacture of nickel-steel eye-bars, these variations between the elastic limit and the ultimate strength of specimens and of full-sized bars will be lessened materially by a careful study of the process of annealing.

Cold-Pressed Threads Test.

An unsuccessful test on bolts of rivet nickel steel with cold-pressed threads was made and repeated. It seemed to be impracticable to manufacture such a bolt so as to have it break in the body before failing in the threaded portion; consequently, it was decided that nickel steel is not a proper material for the fabrication of bolts with cold-pressed threads. It is possible, though, that a nickel steel with a very low carbon content would work satisfactorily.

Tests for Specific Gravity.

Some very careful tests were made of the specific gravities of the two nickel steels and carbon steel, and it was found that the low-nickel steel was 0.017% lighter than carbon steel, and the high-nickel steel 0.043% lighter. As these variations are extremely small, it will be proper to assume the weight of nickel steel to be exactly the same as that of carbon steel, when estimating weights of metal in bridges.

Tests on Torsion.*

Professor W. K. Hatt made for the writer a number of experiments upon rods 1 in. in diameter and 36 in. long, the nickel steel being standard "plate-and-shape" steel, and the carbon steel that ordinarily used in bridgework. He found the modulus of elasticity in torsion to be 13 000 000 for carbon steel and 10 600 000 for nickel steel, the corresponding values of the modulus of rupture being 60 000 and 93 000, showing that low-nickel steel is about 55% stronger than carbon steel in its resistance to rupture, but about 18% inferior to it in resisting distortion from twisting. Figs. 8 and 9 illustrate the results of Professor Hatt's torsion tests.

Specifications for Nickel-Steel Bridges.

From the results of the preceding investigations it is practicable to write specifications for nickel-steel bridges which will possess the same strength, rigidity, and general excellence of design as the best carbon-steel bridges that are being built to-day.

In preparing such specifications, the writer has adopted as a standard those given in "De Pontibus," modifying only those portions which are affected by the change of metal; and, in order to limit the length of this paper as much as possible, he presents here only the modified portions. These are all that are necessary, for the main object of the specifications is to obtain the weights of metal from which the principal diagrams of this paper were prepared. Should anyone desire to design nickel-steel bridges by the use of these specifications, he can do so by combining them properly with those of "De Pontibus." Moreover, if the result of this investigation is to bring nickel-steel bridges into favor, it is the writer's intention to prepare and make public complete specifications for designing such structures.

Metal.—All rolled steel shall be made by the basic open-hearth process.

In bridges composed entirely of nickel steel, the eye-bars, pins, and rollers shall be made of "eye-bar steel"; the rivets, bolts and all adjustable members shall be made of "rivet steel"; and all other portions, except castings, shall be made of "plate-and-shape steel."

* For a complete record of these tests see Appendix E of the original paper in the Society's Library. It contains Professor Hatt's report.

In bridges of mixed nickel and carbon steels, the floor system, the lateral system, and those parts of trusses of minor importance, such as struts having a large excess of section above the theoretical requirements, lacing bars, and stay-plates, may be made of carbon steel; but the floor system shall, preferably, be of nickel steel.

TABLE 4.—Composition of Rolled Steel.

Ingredients.	Percentages.		
	Rivet steel.	Plate-and-shape steel.	Eye-bar steel.
Nickel	3.50 (3.25 to 3.75)	3.50 (3.25 to 3.75)	4.25 (4.00 to 4.50)
Carbon	0.15 (0.12 to 0.18)	0.38 (0.34 to 0.42)	0.45 (0.40 to 0.50)
Phosphorus	0.03 maximum	0.03 maximum	0.03 maximum
Sulphur	0.04 maximum	0.04 maximum	0.04 maximum
Silicon	0.04 maximum	0.04 maximum	0.04 maximum
Manganese	0.60 (0.55 to 0.65)	0.70 (0.65 to 0.75)	0.80 (0.75 to 0.85)

As the manufacturer will have to keep the elastic limit and the ultimate strength up to certain minima, he will be allowed some liberty in the amounts of carbon to use in order to produce the required results, but he is not to attempt to obtain such results by increasing the manganese, nor will he, under any circumstances, be permitted to pass the limits of phosphorus, sulphur, or silicon; in fact, these limits should be kept as much below the specified amounts as practicable, because these elements are all detrimental to the metal. Preferably, the amounts of nickel should be kept within the limits set; but, in case of necessity, the latter, with the written permission of the Engineer, may be varied from.

Method of Determining Elastic Limit.—The elastic limit for specimen tests shall be assumed to be the load on the specimen producing a permanent set of 0.01 in. in a gauge length of 8 in., the amount of set being measured by fine dividers with no load on the specimen, or being taken from an autographic record at the intersection with the stress-strain curve of a line drawn parallel to and 0.01 in. away from the straight portion of the record.

The elastic limit for tests of full-sized eye-bars shall be assumed to be the load on the bar producing a permanent set of 0.04 in. in a gauge length of 20 ft.; or a proportionate set for shorter lengths, the amount of set being measured by an extensometer of approved design, with no load on the eye-bar.

Tensile Strength.—The ultimate tensile strength per square inch on unannealed test pieces for all three kinds of rolled nickel steel used in structural metalwork shall be as follows:

Rivet steel	70 000	to	80 000 lb.
Plate-and-shape steel	105 000	"	120 000 "
Eye-bar steel	115 000	"	130 000 "

The preceding figures are for test pieces taken from the edge of the piece. In case the test pieces are taken from the interior, these figures may be reduced by 3 000 lb. each.

The preceding figures also apply for all plates up to $\frac{7}{8}$ in. in thickness and for all shapes up to $\frac{5}{8}$ in. in thickness. For each additional $\frac{1}{8}$ in. in thickness the ultimate strength may be reduced 1 500 lb. per sq. in., down to an inferior limit of 95 000 lb., for the thickest eye-bar flats.

Elastic Limits.—The least allowable elastic limits per square inch obtained from unannealed test pieces shall be as follows:

Rivet steel	45 000 lb.
Plate-and-shape steel	60 000 "
Eye-bar steel	65 000 "

The preceding figures are for test pieces taken from the edge of the piece. In case the test pieces are taken from the interior, the figures may be reduced by 2 000 lb. each.

The preceding figures also apply for all plates up to $\frac{7}{8}$ in. in thickness and for all shapes up to $\frac{5}{8}$ in. in thickness. For each additional $\frac{1}{8}$ in. in thickness the elastic limit may be reduced 1 000 lb., down to a limit of 57 000 lb. per sq. in. for plate-and-shape and eye-bar steels.

Elongation.—The percentages of elongation shall be obtained from the unannealed test pieces after breaking on an original length of 8 in., in which length must occur the curve of reduction from stretch on both sides of the point of fracture. The least allowable elongations for the three kinds of rolled structural steel shall be as follows:

Rivet steel	25 per cent.
Plate-and-shape steel	15 " "
Eye-bar steel	12 " "

The preceding percentages apply to plates, shapes, and flats $\frac{1}{2}$ in. thick or less. For thicker metal they are to be increased by unity for each increase of $\frac{1}{4}$ in. in thickness.

Bending Tests.—Specimens of rivet steel shall be capable of bending, by either pressure or hammering, to 180° and closing down flat upon themselves without cracking, when either hot or cold.

Specimens of plate-and-shape steel, when either hot or cold, shall be capable of bending by pressure 180° around a mandrel having a diameter equal to twice the thickness of the test piece, without showing signs of cracking on the convex side of the bend.

Specimens of eye-bar steel, when similarly treated, shall be capable of bending by pressure 90° around a mandrel having a diameter equal to three times the thickness of the test piece, without showing signs of cracking on the convex side of the bend.

Drifting Tests.—Punched rivet holes in plate-and-shape steel, pitched two diameters from a sheared edge, must stand drifting until their diameters are 40% greater than those of the original holes, and must show no signs of cracking the metal.

The total taper of the drift-pins used for the testing shall not exceed 1 in 12.

Fracture.—All broken test pieces for all three classes of steel, and all broken eye-bars must show a silky fracture of uniform color.

Full-Sized Eye-Bars.—Full-sized eye-bars must show an ultimate tensile strength per square inch for the various thicknesses of metal as follows:

1-in.	105 000 lb.
1½-in.	100 000 "
2-in.	95 000 "
2½-in. or greater	90 000 "

The elongation shall not be less than 10% in a gauged length of 10 ft.; and the elastic limit shall not be less than 55% of the ultimate strength of the bar.

Cast Steel.—All steel castings shall be made of open-hearth steel of the same composition as that specified for eye-bar steel, except that acid steel having a maximum limit for phosphorus of 0.06% may be used. The ultimate tensile strength shall vary within the limits of 110 000 and 130 000 lb. per sq. in.; the elastic limit shall not be less than 55% of the ultimate tensile strength; and the elongation of test specimens in 2 in. shall not be less than 20 per cent.

Impact Allowance Load.—The impact allowance load is to be a percentage of the equivalent uniform live load found by the following formulas:

$$P = \frac{40\,000}{L + 500} \text{ for railroad bridges,}$$

$$\text{and } P = \frac{10\,000}{L + 150} \text{ for highway bridges,}$$

where P is the percentage and L is the length, in feet, of span or portion of span covered by the live load, when the member considered is subjected to its maximum stress.

Intensities of Working Stresses.—The following intensities of working stresses (that is, pounds per square inch of cross-section) are to be used for all cases, except where wind loads are combined with other loads, in which cases the intensities are to be increased 25%; but the sections shall not be less than those required by the stresses from all loads except wind.

Tension on eye-bars	30 000 lb.
Tension on plates and shapes in bottom chords, main diagonals, and laterals	28 000 "
Tension on net section of plate-girder flanges (assuming one-eighth of the area of the web to act as part of each flange), on extreme fibers of rolled I-beams, and on shapes in body of suspenders, hip verticals, and hanger plates (there being 50% increase of net area for section through eyes)	24 000 "
Bending on pins	50 000 "
Bearing on pins (measured upon the projection of the semi-intrados on a diametral plane)	38 000 "
Bearing on rivets (measured similarly)	30 000 "
Shear on pins	25 000 "
Shear on rivets	14 000 "
Shear on webs of plate girders (gross section)	17 000 "

For field rivets, the intensities for bearing and shear are to be reduced 20 per cent.

Compression on top chords	$30\,000 - 120 \frac{l}{r}$
Compression on inclined end posts	$30\,000 - 140 \frac{l}{r}$
Compression on all other struts with fixed ends	$27\,000 - 120 \frac{l}{r}$
Compression on all other struts with one or two hinged ends	$27\,000 - 160 \frac{l}{r}$

where l is the unsupported length of the strut, in inches, and r is its least radius of gyration, in inches.

Compression on end stiffeners of plate girders..... 22 000 lb.

For forked ends, the intensity of working stress shall be determined by the formula,

$$p = 15\,000 - 450\,\frac{l}{t}$$

where p is the greatest allowable intensity of working stress (impact being considered); l is the unsupported length, in inches, measuring from the center of the pin-hole to the center of the first transverse line of rivets beyond the point at which the full section of the member begins; and t is the total thickness of one jaw, in inches.

The greatest allowable pressure upon expansion rollers of fixed spans, when impact is considered, shall be determined by the equation,

$$p = 1\,000\ d,$$

where p is the permissible pressure, in pounds per linear inch of roller, and d is the diameter of the latter, in inches. The preceding formula is to be used for rollers of swing spans with the span at rest, but, for the span in motion, the formula to be used is

$$p = 400\ d,$$

where d is the mean diameter of the roller, in inches.

In order to anticipate criticism, the writer will now take each feature of the preceding specifications separately and show how the figures it contains were determined.

Composition of Rolled Steel.—The composition of the rivet steel was fixed by the numerous experiments made on various specimens of rivet nickel steel, most of which were objectionable in some particular. The essential requirements for a good rivet nickel steel are the following:

First.—That it shall flow readily, so as to fill the holes properly, and that it shall head without splitting;

Second.—That it is sufficiently soft to permit of the heads being cut off and the rivets being backed out without undue trouble and expense;

Third.—That it shall have sufficient strength as compared with rivet carbon steel to make its use instead of the latter a decided *desideratum.*

The rather wide limits of nickel, carbon, and manganese, of the specification, will assuredly permit manufacturers to fill these requirements.

The composition of the plate-and-shape steel is that specified for the two special melts manufactured for this investigation under the direct supervision of Mr. Colby; and, while it is not claimed that absolute perfection was attained by this first trial, it is not likely that any great improvement in the characteristics of the future plate-and-shape steel, as compared with those of these melts, will be effected. This steel is strong, tough, reliable, and capable of being manipulated in the shops without danger to the workmen, and without unduly great expense.

The satisfactory character of the composition of the eye-bar steel is not so well assured, as it was determined mainly by anticipation, the idea being to obtain as strong and hard a nickel steel as can be worked into eye-bars without running into brittleness.

The existence of a brittle zone in nickel steels has been lately claimed by Dr. H. C. H. Carpenter, Mr. R. A. Hadfield, and Mr. Percy Longmuir, who presented a paper on the subject to the Institution of Mechanical Engineers of England, in November, 1905.*

These gentlemen find that with from 40 to 50 points of carbon, and from 75 to 100 points of manganese, a steel containing 4¼% of nickel has lost none of its toughness or ductility, as compared with steels having smaller percentages of nickel; but that a steel containing 5% of nickel has lost both to a serious extent. Between these two percentages there is probably a well-defined point of demarcation, but exactly where it is, can only be determined by further experiments. These should be made with the least possible delay.

They find also that with 7 or 8% of nickel the brittleness reaches a maximum, at 12% the resilience begins to increase, and at 20% the steel reaches its normal toughness. This is a very curious property of the alloy; and the knowledge acquired by these scientists, if corroborated by further experiments, will prove of great value to all metallurgists who are interested in the manufacture of nickel steel for bridges. The writer has heard, though, from good authority, that the best American practice in the manufacture of nickel steel does not agree with the findings of these experimenters, and that steel contain-

* A résumé of their investigations was published in *The Engineer* (London), November 24th, 1905.

ing 10% of nickel gave excellent results. On account of this disagreement of authorities, it is hoped that the discussion of this paper will bring out such a mass of evidence both *pro* and *con* that the existence or non-existence of the so-called brittle zone will be firmly established.

The recorded properties of the 4¼% nickel steel experimented upon by these Englishmen were as follows:

Carbon	0.40 per cent.
Manganese	0.82 " "
Bending test (on an anvil)	180 degrees.
Yield point in tension	65 500 lb. per sq. in.
Ultimate strength in tension	109 100 " " " "
Elongation in 2 in	20 per cent.
Reduction of area	33 " "
Modulus of elasticity	29 900 000 lb.

The shock test, made by dropping a 46.7-lb. hammer about 14 in., showed that the 4¼% nickel steel absorbed a greater amount of energy before breaking than did any of the other steels tested, the amounts of nickel therein varying from zero to 20 per cent. This shows that, as far as impact is concerned, there is no objection whatsoever to 4¼% nickel steel for eye-bars.

Comparing the properties of this English 4¼% nickel steel with those called for in the preceding specifications for eye-bar steel, it is seen that they strike an average for both nickel and manganese, touch the inferior limit for carbon, exceed somewhat the minimum elastic limit, and fall 6 000 lb. below the inferior limit for ultimate strength. Had the percentage of carbon been 0.45 instead of 0.40, this English steel would undoubtedly have had sufficient ultimate strength to meet the requirements of the specifications.

As for the fitness of 4¼% nickel steel to be manufactured into eye-bars, this is proved by the fact that for this investigation two eye-bars were fabricated from ¾-in. plates of 4¼% nickel steel, as stated previously. These eye-bars contained 46 points of carbon and 67 points of manganese, as determined by the chemist of the Osborn Engineering Company; and the specimen tests gave elastic limits varying from 69 200 to 78 200 lb. (determined by the drop of the beam at medium testing speed), ultimate tensile strengths varying from 112 100 to

122 400 lb., percentages of elongation varying from 11.5 to 18.75 and averaging 16.8, and reductions of area varying (with one exception, where the fracture was unsatisfactory) from 39.5 to 47.1 per cent.

As previously stated, the two eye-bars made from this material gave ultimate strengths of 102 300 and 105 900 lb. per sq. in.; but, unfortunately, the elastic limit in both cases was missed. The elongation in 10 ft. was about 7%, and the reduction of area about 46 per cent.

It must be remembered that these were not *bona fide* eye-bars, for they were manufactured from universal mill plates with planed edges. Had the edges been rolled, instead of planed, it is almost certain that better results would have been found, especially in the elongation. The amount of manganese was less than the specifications call for. Had it been increased, so as to agree with them, the ultimate strength of the specimens would probably not have fallen below their requirements.

As it was, the metal of which these eye-bars were made complied with the specifications in respect to elastic limit, gave an average ultimate strength only 300 lb. per sq. in. below the lowest requirement, and failed only in one case out of fourteen to give the demanded elongation (and then only by one-half of 1 per cent.).

Considering that this was "picked-up" steel, and in view of the great superiority of Mr. Colby's low-nickel steel over the "picked-up" specimens of similar nickel steel, as shown by all the tests, it is fair to conclude that there will be no trouble whatsoever in obtaining eye-bar steel that will easily meet the preceding specifications, and at the same time will permit of satisfactory heads being forged with facility.

The matter of annealing nickel-steel eye-bars, in order to obtain the best possible results, is one worthy of a full and immediate investigation. The writer had intended to settle this question by experiments, but his failure to obtain a special melt of eye-bar steel prevented. It was his intention to test two thicknesses of eye-bars, namely, 2-in., and 2½-in., from 6 to 8 in. wide, and 6 ft. long, in groups of three, by annealing at slightly varying temperatures, in order to determine from the average elastic limits and ultimate strengths of each group what temperature will give the best result for each thickness of metal.

Should this series of tests on nickel steel for bridges ever be continued beyond the limits of this investigation, one of the first steps

would be to manufacture a special melt of eye-bar steel, make the usual specimen tests of the metal, so as to see that it is up to the requirements, prepare a full supply of short eye-bars of various thicknesses, and determine finally the best annealing temperature for each thickness.

The data concerning tests of nickel steel for the eye-bars of the Blackwell's Island Bridge, given in Appendix B, were obtained through the courtesy of the American Bridge Company. From them the writer endeavored to obtain, by averages, approximate rates of variation of elastic limit and ultimate strength with increased thickness of metal; but the attempt was a failure, as the results showed much irregularity, and really indicated that these characteristics of eye-bars are substantially independent of the thickness. Such a conclusion, however, would not be warranted, unless the metal of which all the eye-bars were composed was of practically uniform composition, which it was not, by any means, except in so far as the percentage of nickel was concerned.

In order to obtain, if possible, some general information of value from these Blackwell's Island Bridge tests, the writer has prepared averages of the chemical compositions; of the elastic limits, ultimate strengths, elongations, and reductions of area, of both unannealed and annealed specimens; and of the thicknesses, elastic limits, ultimate strengths, elongations, and reductions of area of full-sized bars, but, in making the averages, he rejected the elongations and reductions of area of those bars that broke in the eye. The results of these computations are as follows:

Chemical Analyses.

Carbon	0.39
Phosphorus	0.012
Sulphur	0.03
Manganese	0.71
Nickel	3.39

Unannealed Specimens.

Elastic limit	60 610 lb. per sq. in.
Ultimate strength	106 385 " " " "
Elongation	17.8% in 8 in.
Reduction of area	29.9%

Annealed Specimens.

Elastic limit	54 377 lb. per sq. in.
Ultimate strength	97 084 " " " "
Elongation	22.5% in 8 in.
Reduction of area	42.9%

Full-Sized Eye-Bars.

Thickness	$1\frac{31}{32}$ in.
Elastic limit	50 006 lb. per sq. in.
Ultimate strength	88 818 " " " "
Elongation	12.2% in 18 ft.
Reduction of area	36.3%

Comparing these average results with the corresponding figures given in the preceding "Specifications for Nickel-Steel Bridges," the following conclusions are reached:

The impurities of phosphorus and sulphur are well within the limits of the specifications, the percentage of carbon is 6 points less, the percentage of manganese 9 points less, and the percentage of nickel 86 points less; but, if this eye-bar metal be compared with the plate-and-shape steel of the specifications, the percentages of carbon and manganese are found to be almost the same, and the percentage of nickel only 11 points less; consequently, the average metal of the Blackwell's Island Bridge eye-bars is almost identical in composition with the plate-and-shape steel of the specifications, and with the special melts made for the writer's experiments.

The average elastic limit and ultimate strength of unannealed specimens are very near the lower limits of the specifications for plate-and-shape steel, and considerably below those for eye-bar steel. Had the specifications for plate-and-shape steel been used for the manufacture of these eye-bars, and had advantage been taken of the clauses permitting the lowering of the elastic limit and ultimate strength with the increased thickness of metal, there would have been no special difficulty in complying with the specifications, as far as the testing of unannealed specimens is concerned.

Had the specifications for eye-bar metal been used for the manufacture of these eye-bars, there would have been numerous rejections because of elastic limit, but very few, if any, on account of ultimate strength, as far as the testing of unannealed specimens is concerned.

As for elongation of unannealed specimens, it is evident that there would have been very little difficulty in complying with the specifications for plate-and-shape steel, and none at all in complying with those for eye-bar steel.

In respect to the tests of full-sized eye-bars, the average ultimate strength fell some 6 000 lb. below the requirements of the specifications, and the average elastic limit about 2 500 lb. below. Of course, in individual instances, the discrepancies were much greater.

Had the eye-bar specifications been drawn on the assumption that plate-and-shape steel was to be used for eye-bars, the limiting ultimate strength for 2-in. bars would have been about 87 500 lb. per sq. in., and the corresponding elastic limit about 48 000 lb. per sq. in. In every test the elastic limit would have been complied with, but in 35% of the tests the ultimate strength would have fallen short, sometimes materially so, owing to the fact that many bars broke in the eye.

As for the elongation, no difficulty would have been found in complying with the specifications.

Comparing the differences in elastic limit and ultimate strength between unannealed and annealed specimens, the averages of all the tests show, respectively, 6 233 and 9 301 lb., the corresponding percentages being 10.3 and 8.7. The writer found in his experiments approximately 9 and 3, which is a fairly close agreement.

Concerning the differences in elastic limit and ultimate strength between unannealed specimens and full-sized eye-bars, the averages of all the tests show, respectively, 10 604 and 17 567 lb., the corresponding percentages being 17.5 and 16.4. The writer found in his experiments approximately 11 and 6. These discrepancies are mainly due to the fact that the inspector made his specimen test slowly, but possibly in part by more careful annealing of the eye-bars. No data were obtained from the American Bridge Company concerning the annealing of their nickel-steel eye-bars, but the writer hopes that some of its officers, in discussing this paper, will treat this point thoroughly.

Method of Determining Elastic Limit.—The method described in the specifications for determining the elastic limit will suffice while the building of nickel-steel bridges is still in its infancy, because at first there will be ample time for testing the new metal; but, afterward, when the manufacture of such structures from nickel steel becomes general, the testing will have to be done on a more commercial

basis, and the elastic limit will then have to be determined by the drop of the beam.

For such conditions, this specification is suggested:

"The elastic limit, in testing specimens, may be determined by the drop of the beam, according to the accepted practice in all steel mills, provided the speed of the machine or the movement of the head is not more than 1 in. in 3 min., up to the elastic limit, and that the weight is moved out by hand at a uniform speed sufficient to keep the beam in very light contact with the upper cross-piece. The speed, after the elastic limit is passed, shall not be greater than 1 in. in 30 sec. nor less than 1 in. in 1 min."

The true elastic limit may be determined as just described, provided the operator maintains a uniform movement of weight, as specified.

Little dependence can be placed on the elastic limits from specimen tests as usually made, because the speed of the machine is so rapid that the lever is kept pressed hard against the upper cross-bar, and thus the elastic limit is passed by 3 000 or 4 000 lb. per sq. in. before the beam has a chance to drop. The ultimate strength is exaggerated in the same manner, but not usually to the same extent as the elastic limit. In order to obtain proper and reliable records for all specimen tests, the limits of the speed, both before and after the elastic limit is passed, should be materially reduced, but not to such an extent as either to involve a hardship for the manufacturer or to cause the inspectors excessive labor or trouble.

Tensile Strength.—It has been shown previously, for this item, that there will be no special difficulty in filling the requirements of the specifications for the three kinds of nickel steel; especially if it be remembered that the figures given as limits may be reduced 3 000 lb. each for specimens taken from the interior, and that there is a sliding-scale reduction for specimens exceeding a certain thickness. With these limitations, manufacturers should have no serious difficulty in obtaining in all cases the ultimate strength called for.

Elastic Limits.—The preceding remarks upon the specification for ultimate strength apply to the elastic limit specification as well.

Elongation.—The specifications governing elongation are by no means difficult to fill; indeed, they are possibly not severe enough, and in the future it may be advisable to raise the percentages somewhat.

Bending Tests.—None of the requirements specified for bending are difficult to satisfy, provided the operator does not attack the metal brutally, but gives it a chance to show what it is worth by taking plenty of time and by seeing that the bend is not made too sharp.

Drifting Tests.—Nor are the drifting-test requirements difficult to satisfy, if the operator will not strike too hard blows and if he will turn the plate over quite often. The enlargement of the holes should always be made gradually and with care.

Fracture.—As practically all the test pieces in this series of tests complied with this requirement, there should be no difficulty in the future in living up to it.

Full-Sized Eye-Bars.—As it is not likely that nickel-steel eye-bars will be made less than $1\frac{1}{2}$ in. or more than $2\frac{1}{2}$ in. thick, it is not probable that there will be any special difficulty in obtaining ultimate strengths of 100 000 lb. for the smaller thickness, and 90 000 lb. for the greater thickness. The writer found, for the weakest of his 2-in. bars, an ultimate strength of 89 000 lb. per sq. in., and, as the metal specified for eye-bar steel is 10 000 lb. stronger in specimens than that of which his eye-bars were fabricated, there ought to be no difficulty in meeting the requirements.

The elastic limit in nickel steel never falls below 55% of the ultimate strength.

Cast Steel.—The specifications for cast steel are not difficult to fill, and if they should prove so in any particular it would be easy enough and perfectly legitimate to amend them, as castings form but a small portion of bridge material.

Impact Allowance Loads.—The impact allowance loads are the same as those given in "De Pontibus," and their reliability has never been questioned; for it is generally conceded that they are decidedly in excess of the real impact. It was the writer's intention, when the book was written, to make them cover, not only the actual impact, but also small, unavoidable secondary stresses and slight inequalities of stress distribution. The various intensities of working stresses adopted were properly adjusted to these impact allowances.

Tension on Eye-Bars.—There are several ways of checking the correctness of the 30 000 lb. per sq. in. for the working stress on eye-bars. One is to take the ratio of least allowed elastic limits for full-sized eye-bars of nickel steel and of carbon steel and multiply it by the

intensity given in "De Pontibus" for carbon-steel eye-bars, namely, 18 000 lb. The result would be:

$$\frac{49\,500 \times 18\,000}{28\,000} = 31\,800 \text{ lb.}$$

Another way is to use the ratio of least allowed ultimate strengths of full-sized eye-bars. The result would then be:

$$\frac{90\,000 \times 18\,000}{56\,000} = 28\,900 \text{ lb.}$$

Still another check is to use the ratio of least allowable elastic limits for specimen tests, which would give:

$$\frac{65\,000 \times 18\,000}{35\,000} = 33\,400 \text{ lb.}$$

Or, adopting the ratio of least allowable ultimate strengths of specimens, there results:

$$\frac{115\,000 \times 18\,000}{60\,000} = 34\,500 \text{ lb.}$$

Averaging these four results gives 32 100 lb., which is well above the 30 000 lb. specified.

Tension on Built Members.—The intensity given in the new specifications is 28 000 lb., and that for carbon steel from "De Pontibus" is 16 000 lb. Applying the ratio of least allowable elastic limits, gives:

$$\frac{60\,000 \times 16\,000}{35\,000} = 27\,400 \text{ lb.}$$

Or, taking the ratio of least ultimate strengths:

$$\frac{105\,000 \times 16\,000}{60\,000} = 28\,000 \text{ lb.}$$

It is true that the first check shows a deficiency of 600 lb., but this small amount is fully compensated for by the greater resistance of nickel steel to the abuse which all metal receives in the shops.

Tension on Net Section of Flanges of Beams, etc.—The intensity of tension on net section specified for nickel steel is 24 000 lb., and that for carbon steel is 14 000 lb. Applying the ratio of least elastic limits gives:

$$\frac{60\,000 \times 14\,000}{35\,000} = 24\,000 \text{ lb.}$$

Bending on Pins.—The ratio of bending resistances on pins for plate-and-shape nickel steel and carbon steel found by the tests was

1.85. As eye-bar steel is about 8% stronger, this ratio should be increased to $1.85 \times 1.08 = 2.0$. The working intensity for bending on pins of carbon steel is 27 000 lb., consequently, that for pins of high-nickel steel would be $2 \times 27\,000 = 54\,000$ lb., while the specifications call for only 50 000 lb. If the ratio of 1.85 were used, the intensity would be almost exactly 50 000 lb.

Bearing on Pins.—For bearing on pins, the writer found, for plate-and-shape steel compared with carbon steel, a ratio of 1.66. The intensity for carbon steel is 22 000 lb., consequently, the application of the ratio would give $22\,000 \times 1.66 = 36\,500$ lb. Owing to the superior stiffness of nickel steel, and because the intensity given in "De Pontibus" is rather low for bearing on pins as compared with the other intensities of the specifications, the intensity for bearing on nickel-steel pins was taken at 38 000 lb. It must be remembered that this is not calculated for the high steel of the pin, but for the lower steel of the bearing.

Bearing on Rivets.—The experiments show that for rivets the ratio of bearing stresses is 1.83. The intensity for carbon steel is 20 000 lb. Applying the ratio gives $20\,000 \times 1.83 = 36\,600$ lb. Another method of calculating this is to take the 38 000 lb. found for bearing on pins and multiply it by the ratio of least elastic limits of rivet nickel steel and plate-and-shape nickel steel, giving $38\,000 \times 45\,000 \div 60\,000 = 28\,500$ lb. As a compromise between these two widely varying results, a value of 30 000 lb. was adopted.

Shear on Pins.—As no experiments were made on shear on pins, the proper intensity can be taken by proportion from the established bending intensities for both nickel-steel and carbon-steel pins and from the given working shear of 15 000 lb. per sq. in. for carbon-steel pins, thus $S = 15\,000 \times 50\,000 \div 27\,000 = 27\,800$ lb. The amount adopted in the specifications is only 25 000 lb.

Shear on Rivets.—The writer found the comparing ratio for ultimate strengths of nickel-steel rivets and carbon-steel rivets in shear to be about 1.4. Applying this to the intensity of working stress for carbon-steel rivets, namely, 10 000 lb., gives 14 000 lb. for the intensity of shear on nickel-steel rivets, which is the figure adopted for the specifications.

Shear on Webs of Plate Girders.—The intensity of working stress for carbon steel is 10 000 lb.; and, as the ratio of elastic limits for

plate-and-shape steel and carbon steel is about 1.7, the intensity for shear on web plates of nickel steel should be 17 000 lb., as given in the specifications.

Compression Formulas.—As the column tests on nickel steel were limited to six, the data for preparing formulas were very meager, but the comparing ratios found by the writer for long and for short columns, namely, 1.47 and 1.75 (the ratios of length to least radius of gyration for the two cases being, respectively, 27 and 81), sufficed for the establishment of empirical formulas similar to those of "De Pontibus."

The short struts tested correspond to top-chord panel lengths, for which the carbon-steel formula is:

$$p = 18\,000 - 70\,\frac{l}{r}.$$

The formula assumed for nickel-steel top chords in the specifications is:

$$p = 30\,000 - 120\,\frac{l}{r}.$$

Testing this for $\frac{l}{r} = 30$, gives

$$p = 15\,900 \text{ for carbon steel,}$$
$$\text{and } p = 26\,400 \text{ for nickel steel.}$$

The ratio of these values is 1.65, instead of 1.75 as shown by the experiments.

For $\frac{l}{r} = 50$, which is the usual limit for top-chord sections of railroad bridges,

$$p = 14\,500 \text{ for carbon steel,}$$
$$\text{and } p = 24\,000 \text{ for nickel steel.}$$

The ratio of these values is 1.65. By interpolation from the experiments, this would have been about 1.64.

These examples show that the new top-chord formula errs on the side of safety for small values of $\frac{l}{r}$, and is just right for the usual values in top chords.

The formula for inclined end posts of nickel steel was obtained approximately from that found for the top chords by simply varying the coefficient of $\frac{l}{r}$ by the ratio of the corresponding coefficients in the formulas for carbon-steel struts.

For all other nickel-steel columns with fixed ends the formula assumed in the specifications was:

$$p = 27\,000 - 120\,\frac{l}{r}.$$

while that for carbon-steel struts is:

$$p = 16\,000 - 60\,\frac{l}{r}.$$

The usual average value of $\frac{l}{r}$ for such struts is about 80, for which the formulas give:

$$p = 17\,400 \text{ for nickel steel,}$$
$$\text{and } p = 11\,200 \text{ for carbon steel.}$$

The ratio of these values is 1.55, which is somewhat more than the ratio given by the experiments for hinged ends; but it would probably be amply safe for fixed ends.

The formula for all other nickel-steel struts with one or two hinged ends was derived from the formula for similar struts with fixed ends by modifying the coefficient of $\frac{l}{r}$ in about the same ratio as exists in the two carbon-steel column formulas, namely, $80 \div 60 = 1.33$. Thus, $120 \times 1.33 = 160$; which is the constant adopted in the nickel-steel column formula for hinged ends.

Testing this for $\frac{l}{r} = 80$ gives:

$$p = 14\,200 \text{ for nickel steel,}$$
$$\text{and } p = 9\,600 \text{ for carbon steel.}$$

The ratio of these values is 1.48, which agrees almost exactly with the results of the experiments.

As soon as nickel steel is actually used for building bridges, it will become necessary to make some additional experiments on columns with both fixed and hinged ends for various lengths so as either to establish some new formulas or to verify the preceding ones. Meanwhile, it will be perfectly safe to use the latter in designing nickel-steel bridges.

Compression on End Stiffeners of Plate Girders.—The intensity for carbon-steel stiffeners is 14 000 lb., and as $\frac{l}{r}$ for end stiffeners is small, say not to exceed 40, the safe ratio to use will be 1.68, hence the intensity for nickel-steel end stiffeners could be $14\,000 \times 1.68 = 23\,500$ lb., while the specifications call for only 22 000 lb.

Formula for Forked Ends.—As $\frac{l}{r}$ in forked ends is likely to run as high as 80, the safe ratio will be about 1.5. Applying this to the carbon-steel formula, namely,

$$p = 10\,000 - 300\,\frac{l}{r},$$

$$\text{gives } p = 15\,000 - 450\,\frac{l}{r}.$$

which is the formula adopted for forked ends of nickel steel.

Formula for Expansion Rollers.—The carbon-steel formula for expansion rollers is $p = 600\ d$. Applying the general ratio of strength, namely, 1.7, gives for the constant $1.7 \times 600 = 1\,040$; hence the formula for nickel-steel rollers at rest was made $p = 1\,000\ d$.

For rollers in motion, this was changed to $p = 400\ d$.

The only specification previously written for nickel steel in bridges, so far as the writer knows, is that of R. S. Buck, M. Am. Soc. C. E., for the Manhattan Bridge, issued by the Department of Bridges of the City of New York. In it the following intensities are specified for nickel-steel members for a combination of dead load, temperature, and congested live load, or for a combination of dead load, regular live load, temperature, and wind.

	Pounds per square inch.
Tension in stiffening trusses	40 000
Compression in stiffening trusses	$40\,000 - 150\,\frac{l}{r}$
Shear on rivets in stiffening trusses (field)	20 000
Bearing on rivets in " " "	35 000

As the writer's proposed specifications are for a combination of dead load, live load, impact-allowance load, and wind load, his allowance for impact, for the purpose of comparison, may be allowed to offset Mr. Buck's allowance for temperature. For this combination, the writer's intensities of working stresses would be as follows:

	Pounds per square inch
Tension in stiffening trusses (eye-bars)	37 500
Tension in stiffening trusses (shapes)	35 000
Compression in stiffening trusses (chords)	$37\,500 - 150\,\frac{l}{r}$
Compression in stiffening trusses (webs), fixed ends	$33\,750 - 150\,\frac{l}{r}$

Compression in stiffening trusses (webs), hinged ends	33 750	$200\ \frac{l}{r}$
Shear on rivets in stiffening trusses (field)	14 000	
Bearing on rivets in stiffening trusses (field)	30 000	

Comparing the two sets of figures, it will be seen that Mr. Buck has in every case strained his nickel steel higher than the writer has strained his, notwithstanding the fact that the requirements for strength of metal in Mr. Buck's specifications are decidedly less than in those of the writer. It is evident, therefore, that, as compared with the *dicta* of the sole present authority on the use of nickel steel in bridgework, the specifications herein presented for the designing of nickel-steel bridges err assuredly on the side of safety. It is true that Mr. Buck's specifications are for a very long span, and in consequence his intensities of working stresses are permitted to run high by the best established engineering practice; but it must be remembered that the writer's intensities, by the impact allowance of his specifications, are adjusted properly for spans of all lengths.

In making this comparison the writer is not endeavoring to criticise Mr. Buck's specifications, but is simply anticipating possible criticism of his own on the plea of overstraining the nickel steel.

On Figs. 10 to 21, inclusive, are given the weights of all ordinary single-track and double-track railway bridges for all spans up to 1 800 ft. There are included four types of cantilever bridges, namely, *A*, *B*, *C*, and *D*, as shown on Fig. 22. Type *A* is the most usual, and is suitable where only one long span is necessary. Type *B* is for a bridge of very great length, where two long main spans are required. Type *C* is for the case where the total length is greater than for Type *A*, but short as compared with that for Type *B*. As regards economy of metal, Type *C* comes next to Type *A*. Type *D* is intermediate between Types *B* and *C*. These four types cover all the possible layouts of spans for cantilever bridges, or at least all that are consistent with good engineering practice.

Class *R* of the "De Pontibus" specifications was adopted as the live load for simple-span bridges; and, for cantilever structures, Class *R* was used for the stringers, Class *S* for the floor beams and the primary truss members, and Class *U* for the main truss members. All the cantilever structures were assumed to have double tracks, as such bridges are now rarely, if ever, built with single tracks.

For the purpose of record, all lay-outs for cantilever bridges were assumed to have the following constant proportions between the lengths of their various spans:

Calling l the length of the main span of Type *A*, $\frac{3}{8}\ l$ will be the length of the suspended span and $\frac{5}{16}\ l$ that of each cantilever arm and of each anchor arm. For the anchor span, when there is one, the length will be $\frac{5}{8}\ l$. These proportions are all shown correctly to scale on Fig. 22. The plotted weights of metal per linear foot of span for the cantilever bridges are the average weights for the entire length of structure.

The weights of metal per linear foot of span for the carbon-steel bridges were computed by using the specifications of "De Pontibus," and those for the nickel-steel and mixed-steel bridges by the specifications of this paper combined with those of "De Pontibus." These weights are as accurate as they can well be made, and much time was spent by the writer's office force in calculating them. At some future time, after bridge building in nickel steel has been inaugurated, the writer will give to the Profession curves of weights of metal per linear foot in nickel-steel, mixed-steel, and carbon-steel bridges for all his standard live loads; but, for the present, those offered in this paper will have to suffice.

From the weights of metal per linear foot, given on Figs. 10 to 21, inclusive, from various assumed pound prices of carbon-steel bridges erected, and from various assumed differences in pound prices of superstructure metal delivered at site, in nickel steel and in carbon steel, the costs in dollars per linear foot of span (for bridges of all types in "all nickel steel," "mixed steel," and "carbon steel") were plotted, and are given in Figs. 23 to 72, inclusive.*

The range of pound prices for carbon steel erected is from 2.5 to 5.5 cents for plate-girder bridges, and from 3.0 or 3.5 cents to 6.0 cents for all other bridges. These ranges are likely to include, for many years, all the conditions of the market for carbon-steel bridges erected: although a combination of general prosperity and a distant and difficult location might cause the superior limits to be passed occasionally. Such a contingency, however, is too remote to warrant the preparation

* Only 50 of 154 diagrams are reproduced in this paper. The others may be seen in the Society's Library. These 50, however, are all that will usually be needed in the immediate future when comparing the costs of carbon-steel bridges with those of bridges built of nickel steel in all parts where the adoption of the alloy is economical, and of carbon steel in all other parts.

of more diagrams than those that accompany this paper, especially since, in such a case, it would require only a few minutes to make by proportion the necessary correction for the special proposed structure.

The differences in pound prices of nickel steelwork and carbon steelwork delivered at site range from 0.6 cent to 2.0 cents. These ought to be sufficient, for when the difference becomes as low as 0.6 cent, these tables will have served their purpose and become obsolete, because then practically all bridges will be built of nickel steel; and even to-day a variation of 2 cents would indicate an excessive overcharge on the part of the manufacturers. In the case of the Manhattan Bridge, the difference bid by the contractors between nickel-steel eye-bars and carbon-steel eye-bars erected was 1.5 cents per lb.; consequently, the difference for the steels delivered at site would have been somewhat less than that amount. It is true that the difference in cost of manufacture of entire bridges in nickel steel and in carbon steel is somewhat greater than the corresponding difference in the case of eye-bars alone, but the variation is certainly not so great as $\frac{1}{2}$ cent per lb. If a greater difference than 2 cents per lb. between the values of nickel steelwork and carbon steelwork delivered at site should occur in any case, it would be an easy matter to plot a curve to meet the condition by proportionate extension on the special diagram that is to be used.

The plotted pound prices for carbon-steel bridges erected vary by $\frac{1}{2}$ cent per lb. In case it is desired to assume any price intermediate between those on the diagrams, the comparison between costs per foot of nickel-steel, mixed-steel, and carbon-steel bridges erected should be made, first, for the next greater price and then for the next lower one, after which the necessary interpolation would be a simple matter.

The proportion of the total cost of the erected metal which pertains to the erection has been arbitrarily assumed for convenience at 20 per cent. This gives a range of from 0.5 cent to 1.2 cents per lb. as the cost of erection, which is a fair assumption, for, even in elevated-railroad work, the cost of erection and painting is seldom as low as $\frac{1}{2}$ cent per lb., and for railroad bridges, even in remote localities, it does not often exceed 1.2 cents per lb. For cantilever bridges, the cost of erection might be more, and thus a comparison of cost slightly too favorable to nickel steel might be made, were it not for the fact that in such large structures the use of this metal will have a tendency to

lower comparatively the pound cost of erection, because the decrease in weight of individual members facilitates progress and reduces the cost of the traveler, derricks, and other heavy apparatus. On the whole, the assumption made for the proportionate cost of erection is, perhaps, the fairest that could be adopted.

In comparing the costs of erection of nickel-steel bridges and carbon-steel bridges due cognizance was taken of the fact that, for two similar bridges of equal carrying capacity, while the total cost of erection of the nickel-steel structure is less than that for the carbon-steel structure, the cost per pound in the former is greater than in the latter, because certain items of expense are constant while others vary with the weight of metal handled. The writer has assumed that one-half the total expense is constant and that the other half will vary directly with the weight of metal. This is as accurate a division as can be assumed. Upon this basis was established the following mathematical statement:

Let W = weight of metal per linear foot of span in the carbon-steel bridge,

W' = ditto for the nickel-steel bridge,

C = cost per pound for erecting the carbon-steel bridge,

C' = ditto for the nickel-steel bridge,

F = cost per linear foot for erecting the nickel-steel bridge,

then CW = cost per linear foot for erecting the carbon-steel bridge,

$$F = \frac{CW}{2}\left(1 + \frac{W'}{W}\right).$$

$$C' = \frac{F}{W'} = \frac{CW}{2W'}\left(1 + \frac{W'}{W}\right) = \frac{C}{2}\left(\frac{W}{W'} + 1\right).$$

In plotting the curves of cost of nickel-steel bridges and mixed-steel bridges erected, given on the diagrams, the cost per pound of the erection of the alloy was computed by this last equation.

The types of bridges covered by the diagrams are as follows:

Single-track, deck, plate-girder spans,

Single-track, half-through, plate-girder spans,

Single-track, through, riveted, Pratt-truss spans,

Single-track, through, pin-connected, Pratt-truss spans,

Single-track, through, pin-connected, Petit-truss spans,
Double-track, through, riveted, Pratt-truss spans,
Double-track, through, pin-connected, Pratt-truss spans,
Double-track, through, pin-connected, Petit-truss spans,
Double-track, through, pin-connected, "Type *A*," cantilever bridges,
Double-track, through, pin-connected, "Type *B*," cantilever bridges,
Double-track, through, pin-connected, "Type *C*," cantilever bridges,
Double-track, through, pin-connected, "Type *D*," cantilever bridges.

These twelve types (barring double-track, plate-girder spans) cover practically all the bridges that are built nowadays in the United States. In the case of any type not included in the preceding list, the diagrams may be used by adopting that one for the structure most like it and for the existing conditions of the metal market. The differences between Types *A*, *B*, *C*, and *D* of cantilever bridges, illustrated on Fig. 22, were explained previously. In the diagrams of weights of these cantilever bridges it must not be forgotten that the weights of metal per linear foot of bridge given are the averages from end to end of structure, and are not the weights per foot for any particular span or spans.

In computing the weights of metal for various spans which are plotted on Figs. 10 to 21, inclusive, it was found, as might readily have been anticipated, that the economic truss and girder depths are somewhat less for nickel-steel than for carbon-steel bridges. The reasons for this are: first, that, in comparison with carbon steel, nickel steel can be strained higher in the compression members of chords than in those of webs; and, second, that in the webs of nickel-steel bridges there are necessarily more minimum sections used than in those of carbon-steel bridges.

It is evident that, on account of both the smaller economic depths and the higher intensities of working stresses, the deflections of nickel-steel spans will be greater than those of corresponding carbon-steel spans. However, this increase of deflection is not a matter of any great importance.

If, eventually, nickel steel should supplant carbon steel in bridge-work, the latter metal will continue to be used for a long time in parts that do not take direct stress (such as stay-plates, lacing bars, and web stiffeners), in the lateral systems of all bridges, except those of extremely long spans; and in truss members having much larger sections than the stresses call for, such as web members near mid-span and the secondary vertical posts of Petit trusses.

In some cases, especially when bridge metal is cheap, a still further saving might be effected by making the entire floor system of carbon steel; but as the amount of money thus gained in the floor would be small, and as it would have to be reduced somewhat by the increased cost of trusses due to the slightly greater dead load, this kind of economy is problematical. In long-span bridges the necessity of keeping the dead load as low as possible would preclude the adoption of carbon steel for the floor system, even if the use of such steel there were *per se* decidedly the cheaper.

In all the diagrams it has been assumed that carbon steel would be used exclusively for lateral systems; but it is a fact that in long spans it would be economical to adopt nickel steel for some of the heavier lateral members, consequently, the comparative costs of long-span bridges, of carbon-steel and of mixed-steel, in case of actual designs with careful detailing, might show even greater differences than those given by the curves.

In order to demonstrate how the diagrams are to be used, it will be well to assume a few cases and apply the curves to their solution.

Case 1.—A long, single-track bridge consists of a succession of half-through, plate-girder spans of 100 ft. each, carbon steel erected costing 4.5 cents per lb., and nickel steel delivered at site being worth 1.6 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 31, there are found $101.50 as the cost per linear foot of the metal in the carbon-steel bridge, and $92 as the corresponding cost for the mixed-steel bridge.

Case 2.—A double-track bridge consists of four riveted, through spans of 200 ft. each, carbon steel erected costing 4 cents per lb., and nickel steel delivered at site being worth 1.4 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 51, there are found $188 as the cost per linear foot of the metal in the carbon-steel bridge, and $174 as the corresponding cost for the mixed-steel bridge.

Case 3.—A double-track, Type *A*, cantilever bridge has a main span of 1 050 ft. If built of carbon steel, it would cost 5.5 cents per lb. erected. Nickel steel delivered at site is worth 1.5 cents per lb. more than carbon steel. Find the comparative costs of carbon-steel and mixed-steel bridges.

Turning to Fig. 69, there are found the following:

Carbon-steel bridge	$658
Mixed-steel bridge for excess of 1.6c	555
" " " for excess of 1.4c	543
By interpolation for excess of 1.5c	549

An attempt will be made to anticipate what will be the probable excess pound price for shopwork on nickel steel as compared with carbon steel, using as a basis the present average cost of reamed shopwork, which is approximately $\frac{8}{10}$ cent per lb. in the principal American bridge shops. This figure includes all expenses and fixed charges of every kind, such as heat, light, power, and office expenses.

To obtain a result that is closely accurate, it will be necessary to itemize the various shop costs for carbon-steel work, add to each item its *pro rata* share of general expense, determine for each item the approximate ratio of increase for nickel-steel work, and calculate the increased items and increased total cost per pound.

The division of cost given in Table 5 is probably as good an average as could be assumed; but it must be remembered that any division whatsoever would vary with the style and individuality of the shops, with the character of the construction, and even with the personnel of the shop management. Table 5 gives all the information required for obtaining the cost of reamed shopwork on nickel steel for bridges.

Each of the ratios of increased cost given in Table 5 was carefully considered in consultation with John Lyle Harrington, M. Am. Soc. C. E., and the items of division of shop cost were furnished by him.

Table 5 shows that the excess cost per pound for the manufacture of nickel-steel bridges, as compared with carbon-steel bridges, is 0.15 cent. Curiously enough, this is exactly the figure named to the writer four years ago as an off-hand guess by C. C. Schneider, Past-President, Am. Soc. C. E.

TABLE 5.—Comparison of Cost of Shopwork per Pound for Carbon Steel and Nickel Steel.

Items.	Cost of shop-work per pound for carbon steel.	Ratio of increased cost for nickel steelwork.	Cost of shop-work per pound for nickel steel.
Drawing-room work	0.08 cent.	1.25	0.100 cent.
Template-shop work	0.04 "	1.25	0.050 "
Laying-out work	0.04 "	1.10	0.044 "
Shearing and straightening	0.04 "	1.10	0.044 "
Punching	0.08 "	1.25	0.100 "
Assembling and bolting	0.12 "	1.10	0.132 "
Reaming and drilling	0.15 "	1.10	0.165 "
Chipping and milling	0.02 "	1.50	0.030 "
Riveting	0.16 "	1.30	0.208 "
Painting	0.03 "	1.25	0.037 "
Miscellaneous	0.04 "	1.00	0.040 "
Total and average	0.80 "	1.19	0.950 cent.

Nickel-steel ingots, when nickel is worth 30 cents per lb., cost about 1 cent per lb. more than those of carbon-steel; and the difference in cost of rolling should certainly not exceed $\frac{1}{10}$ cent per lb. Allowing 20% profit on these excess costs would make the total excess cost of nickel-steel bridgework delivered at site 1.5 cents per lb. The almost exact agreement of this difference with that for the eye-bars in the Manhattan Bridge appears to be pretty conclusive.

Adopting, then, 1.5 cents as the probable difference in pound prices of nickel steelwork and carbon steelwork delivered at site, it will be interesting to compare the costs of bridges of carbon-steel and of mixed-steel of all the twelve kinds covered by the diagrams.

In making this comparison, it will be assumed that the average pound prices for carbon-steel bridges erected throughout the United States are as follows:

Plate-girder spans	4.0 cents.
Riveted-truss spans	4.5 "
Pin-connected, Pratt-truss spans	4.5 "
Pin-connected, Petit-truss spans	5.0 "
Cantilever bridges	5.5 "

The reason for the greater assumed pound costs of long-span bridges is mainly expensive erection, because such spans are generally used where the erection conditions are costly.

From Fig. 25 it is found that for deck, plate-girder spans, carbon-steel bridges are cheaper than mixed-steel bridges only for spans of less than 33 ft., and that in all greater spans they are more expensive. A study of Figs. 30, 36, 41, 47, 52, 57, 63, and 69 shows that, for the conditions assumed, carbon-steel bridges are invariably more expensive than those of mixed nickel and carbon steels. The percentages of the greater cost are given in Table 6.

TABLE 6.—PERCENTAGES OF EXCESS OF COST OF CARBON-STEEL BRIDGES OVER MIXED-STEEL BRIDGES.

Type of Structure.	Least.	Greatest.	Approximate average.
Single-track, deck, plate-girder spans	− 5	− 11	− 5
Single-track, half-through, plate-girder spans	+ 3	+ 12	+ 7
Single-track, through, riveted, Pratt-truss spans	+ 2	+ 7	+ 5
Single-track, through, pin-connected, Pratt-truss spans	− 1	+ 11	+ 6
Single-track, through, pin-connected, Petit-truss spans	+ 10	+ 17	+ 14
Double-track, through, riveted, Pratt-truss spans	+ 4	+ 8	+ 6
Double-track, through, pin-connected, Pratt-truss spans	+ 2	+ 11	+ 6
Double-track, through, pin-connected, Petit-truss spans	+ 13	+ 20	+ 16
Cantilever bridges of "Type *A*"	− 7	+ 30	+ 18
Cantilever bridges of "Type *B*"	− 9	+ 29	+ 19
Cantilever bridges of "Type *C*"	− 7	+ 25	+ 18
Cantilever bridges of "Type *D*"	− 12	− 26	− 20
General average for all bridges	+ 5	+ 17	+ 12

Summarizing, it is evident that it would be economical at the present time to use nickel steel for all kinds of railroad bridges, and the longer the spans the greater the economy. It might be shown also that nickel steel would be economical for certain highway bridges, but its adoption would certainly be inadvisable for ordinary county bridges, because the use of the new metal might cause such structures to "vanish into thin air."

The general use of nickel steel for bridges not only would result in decidedly cheaper structures, but also would permit of the building of longer spans than are at present attainable.

For instance, it is generally conceded by bridge engineers that the present greatest practicable main-span length for cantilevers built of

carbon steel is in the neighborhood of 2 000 ft. On Fig. 71 are plotted the probable weights of metal per linear foot of bridge in carbon steel and in nickel steel for main spans far longer than any yet designed or computed. The method adopted for plotting was to record the already diagrammed weights for spans of 1 200 ft., 1 350 ft., 1 500 ft., 1 650 ft., and 1 800 ft., pass through the five points of each record a circular curve, and carry that curve to the limits of the paper. The weights of metal thus established for spans of unprecedented length are probably fairly accurate. In any case they are sufficiently so for the present purpose, which is simply to determine approximately the main span lengths of cantilever bridges of "Type *A*" that have the same average weight of metal per linear foot for the two kinds of steel. The diagram gives the following corresponding main-span lengths:

Carbon-steel bridges.....	1 200 ft.	1 400 ft.	1 600 ft.	1 800 ft.	2 000 ft.
Nickel-steel bridges.....	1 650 "	1 830 "	2 040 "	2 300 "	2 600 "

From this it will be seen that if 1 800 ft. be assumed as the present practicable limit of span length for carbon-steel bridges, the corresponding limit for nickel-steel bridges will be about 2 300 ft.; or, if it be assumed at 2 000 ft., the corresponding limit for nickel-steel construction will be 2 600 ft. It is safe, therefore, to conclude that the adoption of nickel steel for bridges would lengthen the practicable span length for cantilevers fully 500 ft. The writer foretold this result just before the experiments on nickel steel for bridges were inaugurated.

If the question of greatest span length be one of economics instead of practicability, the curves on Fig. 72 should be used. These were prepared from the data on Fig. 69, assuming that the excess value of nickel steel delivered at site is 1.5 cents per lb.

The diagram gives the following corresponding lengths of main spans for equal costs per linear foot of bridge:

Carbon-steel bridges.....	1 200 ft.	1 400 ft.	1 600 ft.	1 800 ft.	2 000 ft.
Nickel-steel bridges.....	1 406 "	1 660 "	1 800 "	2 010 "	2 265 "

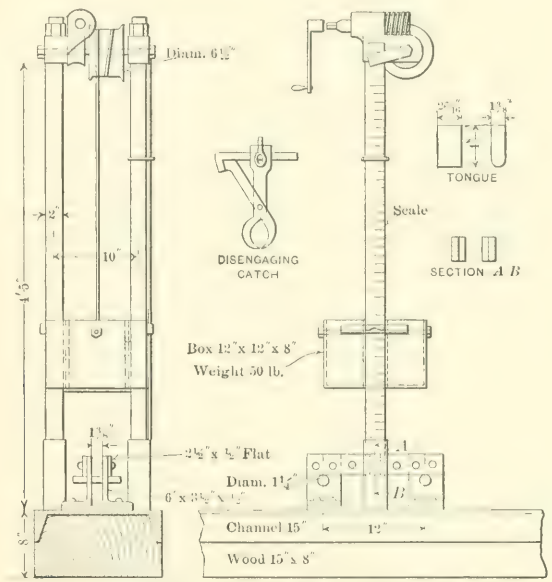
The principal application of the results of this last investigation is to the comparative economy of cantilever and suspension bridges; for if it be known that for carbon-steel construction the length of cantilever main span corresponding to equal cost per foot be 2 000 ft., it can be seen from the table that for nickel-steel construction it is 2 265 ft.

Summarizing the results of this entire investigation of nickel steel for bridges, it is evident that nickel steel is in every way fitted for bridge construction, in that it is strong, tough, workable, and reliable; moreover, its adoption would effect a decided economy. This economy would increase in the future as the cost of nickel decreases and as the shops become more accustomed to the fabrication of the new alloy. That nickel will soon be less expensive is a foregone conclusion, in view of the immense deposits of nickel ore that have been located and surveyed in Canada. It is said upon good authority that there has been found in one deposit in that country ore containing fully 200 000 tons of the metal.

While the writer has never known nickel to have been sold for less than 30 cents per lb., nevertheless, he is of the opinion that, should this material be called for in large tonnages for bridge building, it might be purchased as low as 25 cents. It makes a great difference in the price to the producer whether a metal is sold by the pound or by the ton; and tons of nickel would be required where pounds are bought to-day, were nickel steel used extensively for bridgework.

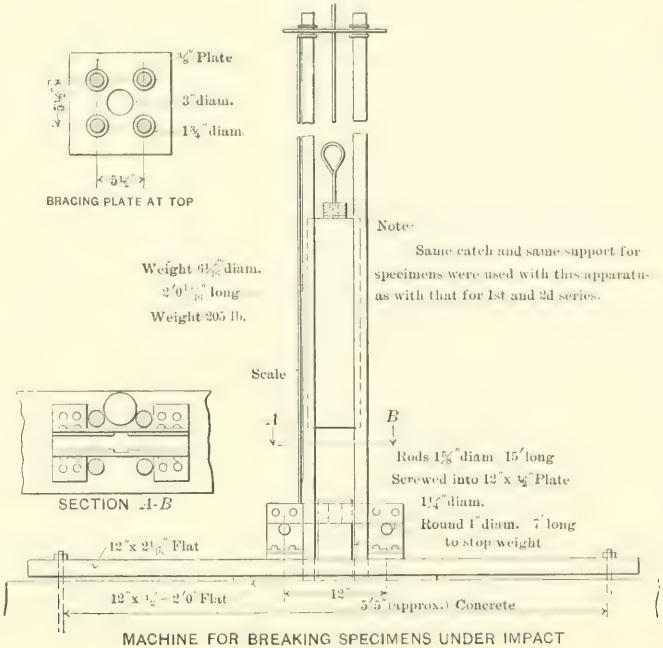
At 25 cents per lb. for nickel, the price of rolled nickel steel would be about 0.2 cent per lb. lower than it would be with nickel at 30 cents per lb., and it is likely that the cost of manufacture would be reduced in the same proportion, thus making the price of nickel-steel bridge metal delivered at site 1.2 cents per lb. above that of carbon steelwork. A study of the accompanying diagrams will show the great economy of using nickel steel for bridges under such conditions.

In concluding this paper the writer desires to ask for a thorough discussion, and to express the hope that the effect of the paper and the discussion will be to hasten materially the adoption of nickel steel for bridges.



MACHINE FOR BREAKING SPECIMENS UNDER IMPACT
1ST AND 2D SERIES

FIG. 1.



MACHINE FOR BREAKING SPECIMENS UNDER IMPACT
3D SERIES

FIG. 2.

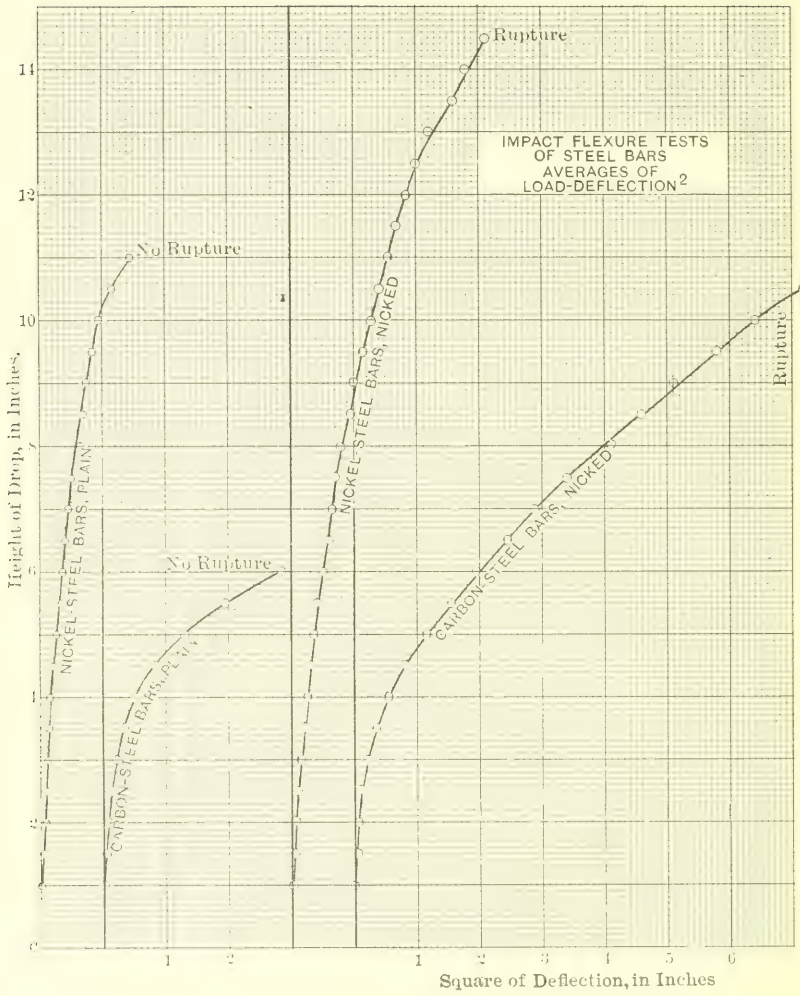

IMPACT FLEXURE TESTS
OF STEEL BARS
AVERAGES OF
LOAD-DEFLECTION²
Rupture
No Rupture
No Rupture
Rupture
NICKEL-STEEL BARS, PLAIN
CARBON-STEEL BARS, PLAIN
NICKEL-STEEL BARS, NICKED
CARBON-STEEL BARS, NICKED
Height of Drop, in Inches.
Square of Deflection, in Inches
14
12
10
1
2
3
4
5
6

FIG. 3.

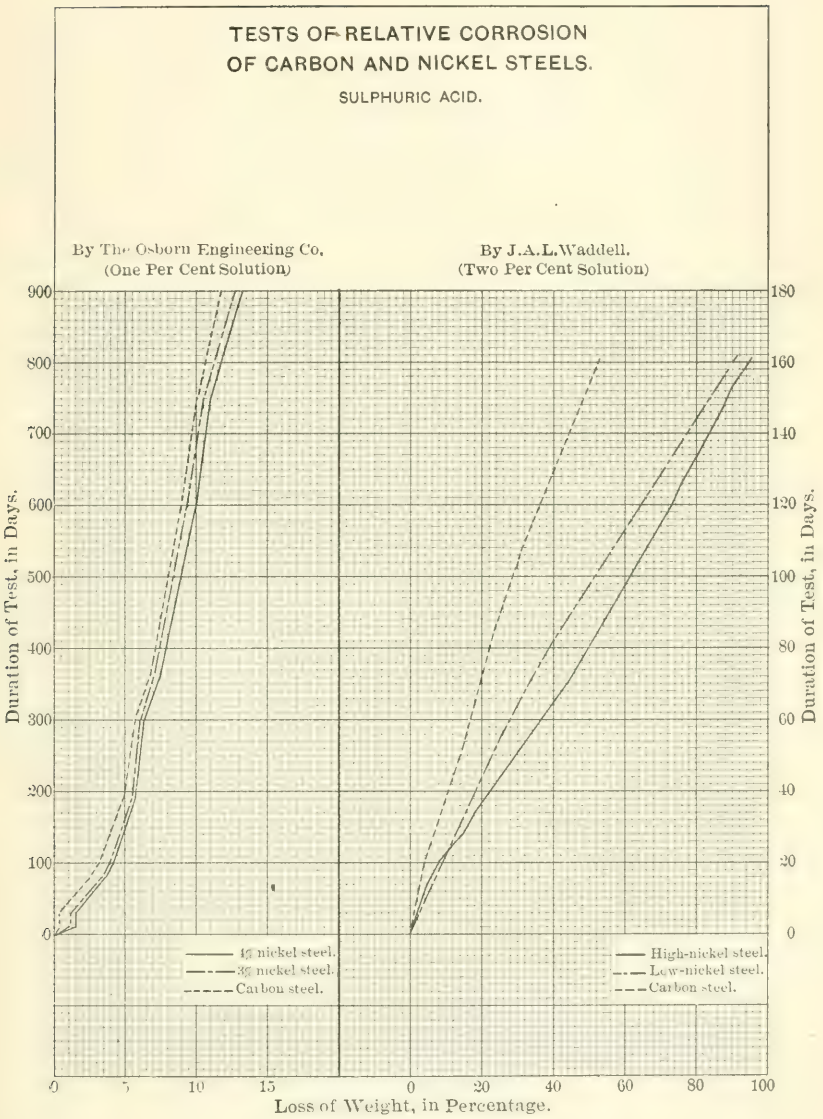

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
SULPHURIC ACID.
By The Osborn Engineering Co.
(One Per Cent Solution)
By J.A.L.Waddell.
(Two Per Cent Solution)
Duration of Test, in Days.
900
800
700
600
500
400
300
200
100
0
180
160
140
120
100
80
60
40
20
0
0
5
10
15
0
20
40
60
80
100
Loss of Weight, in Percentage.
1% nickel steel.
3½ nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.

FIG. 4.

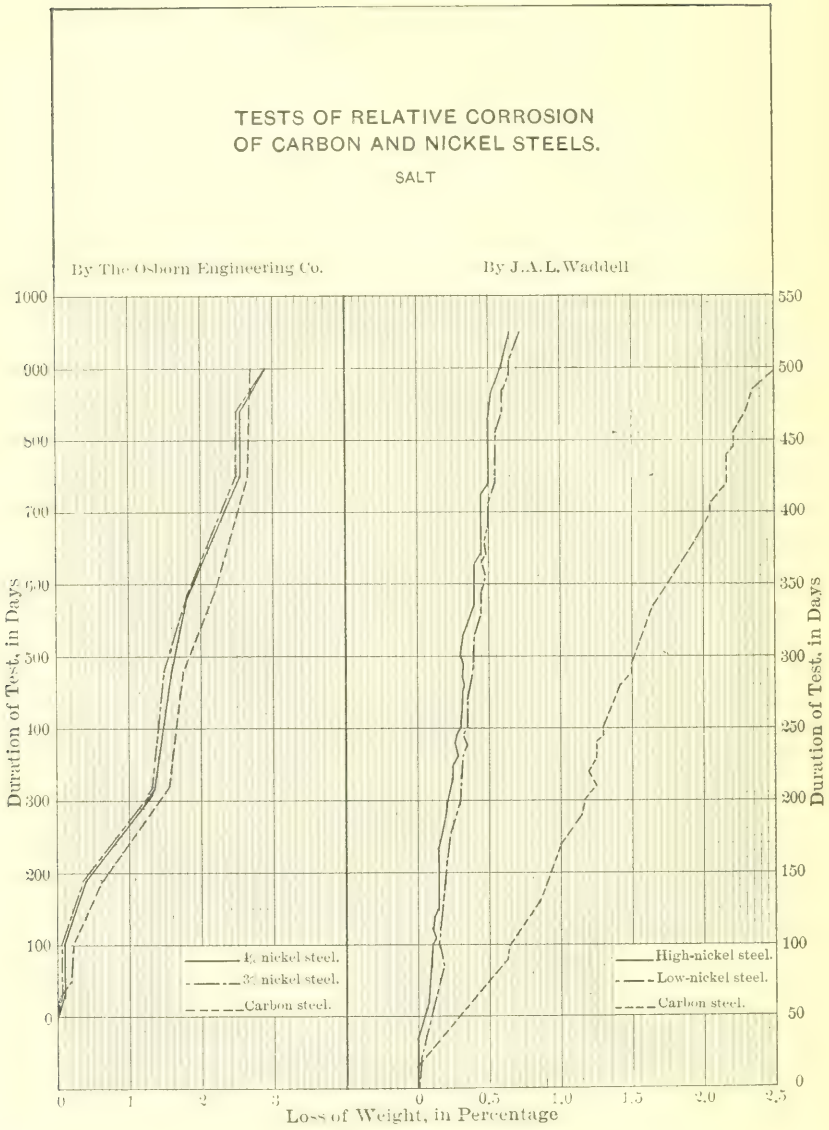

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
SALT
By The Osborn Engineering Co.
By J. A. L. Waddell
Duration of Test, in Days
Duration of Test, in Days
Loss of Weight, in Percentage
1000
900
800
700
600
500
400
300
200
100
0
550
500
450
400
350
300
250
200
150
100
50
0
0
1
2
3
0
0.5
1.0
1.5
2.0
2.5
½ nickel steel.
¾ nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.

FIG. 5.

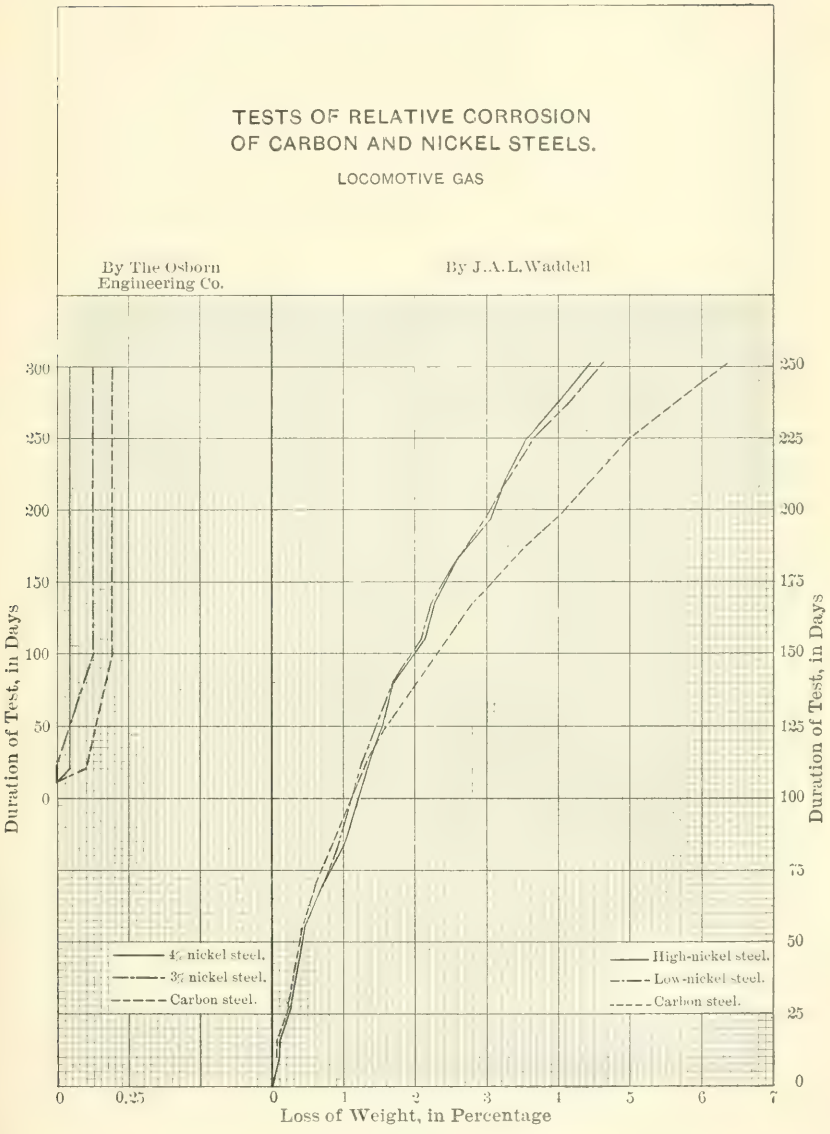

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
LOCOMOTIVE GAS
By The Osborn Engineering Co.
By J.A.L.Waddell
Duration of Test, in Days
300
250
200
150
100
50
0
250
225
200
175
150
125
100
75
50
25
0
0
0.25
0
1
2
3
4
5
6
7
Loss of Weight, in Percentage
4% nickel steel.
3½ nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.

FIG. 6.

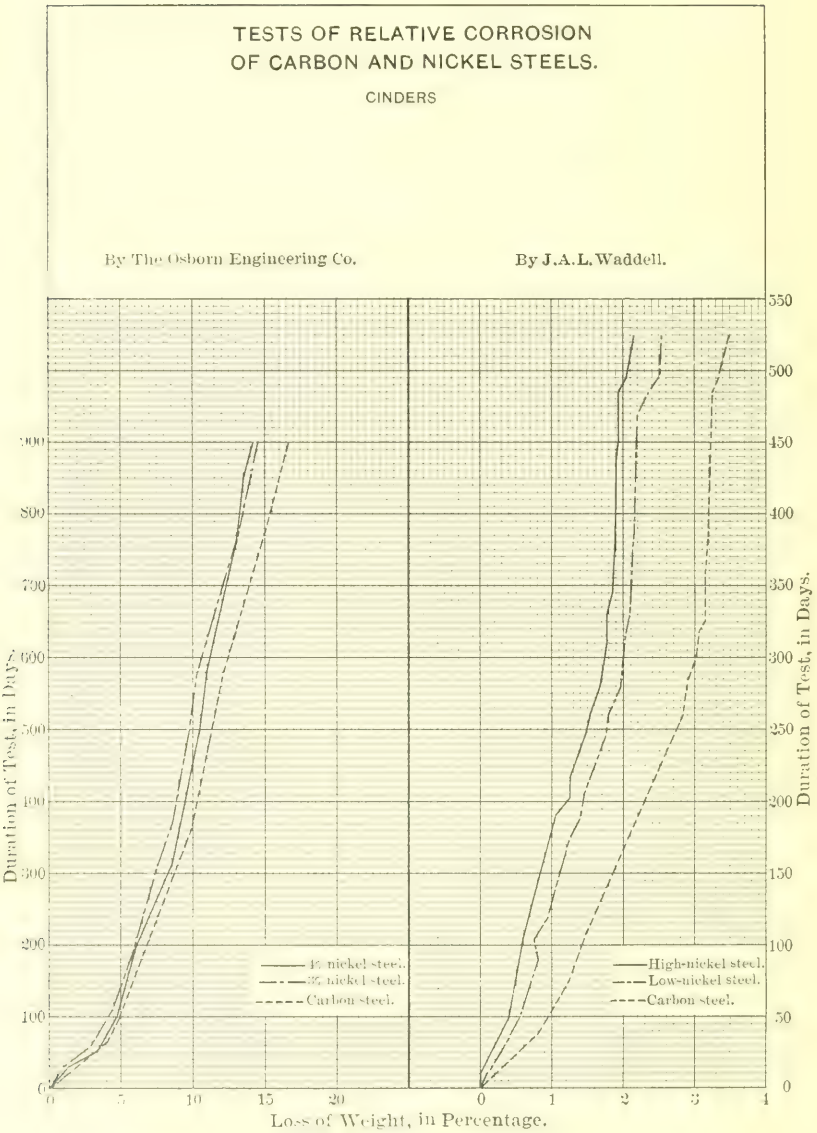

TESTS OF RELATIVE CORROSION
OF CARBON AND NICKEL STEELS.
CINDERS
By The Osborn Engineering Co.
By J.A.L. Waddell.
Duration of Test, in Days.
Loss of Weight, in Percentage.
1% nickel steel.
3½ nickel steel.
Carbon steel.
High-nickel steel.
Low-nickel steel.
Carbon steel.

FIG. 7.

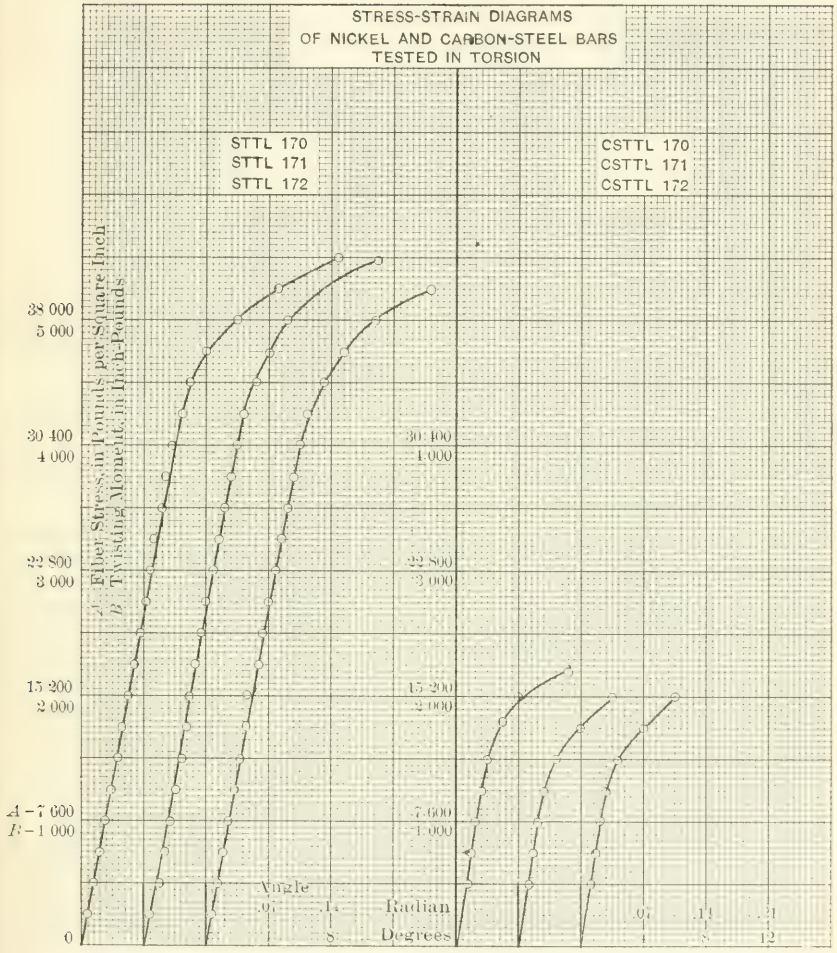

STRESS-STRAIN DIAGRAMS
OF NICKEL AND CARBON-STEEL BARS
TESTED IN TORSION
STTL 170
STTL 171
STTL 172
CSTTL 170
CSTTL 171
CSTTL 172
Fiber Stress, in Pounds per Square Inch
Twisting Moment, in Inch-Pounds
38 000
5 000
30 400
4 000
22 800
3 000
15 200
2 000
A – 7 600
B – 1 000
0
30 400
4 000
22 800
3 000
15 200
2 000
7 600
1 000
Angle
Radian
Degrees

FIG. 8.

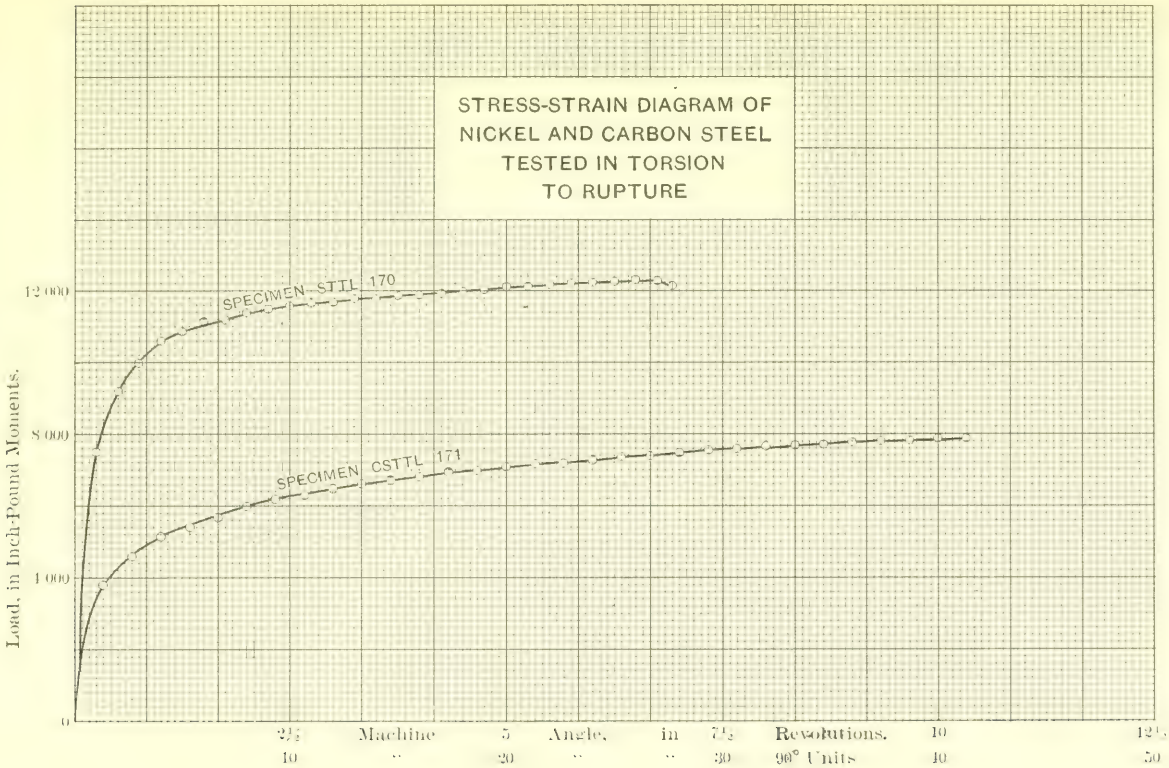

STRESS-STRAIN DIAGRAM OF
NICKEL AND CARBON STEEL
TESTED IN TORSION
TO RUPTURE
SPECIMEN STTL 170
SPECIMEN CSTTL 171
Load, in Inch-Pound Moments.
12 000
8 000
4 000
0
2½ Machine 5 Angle, in 7½ Revolutions. 10 12½
10 " 20 " " 30 90° Units 40 50

FIG. 9.

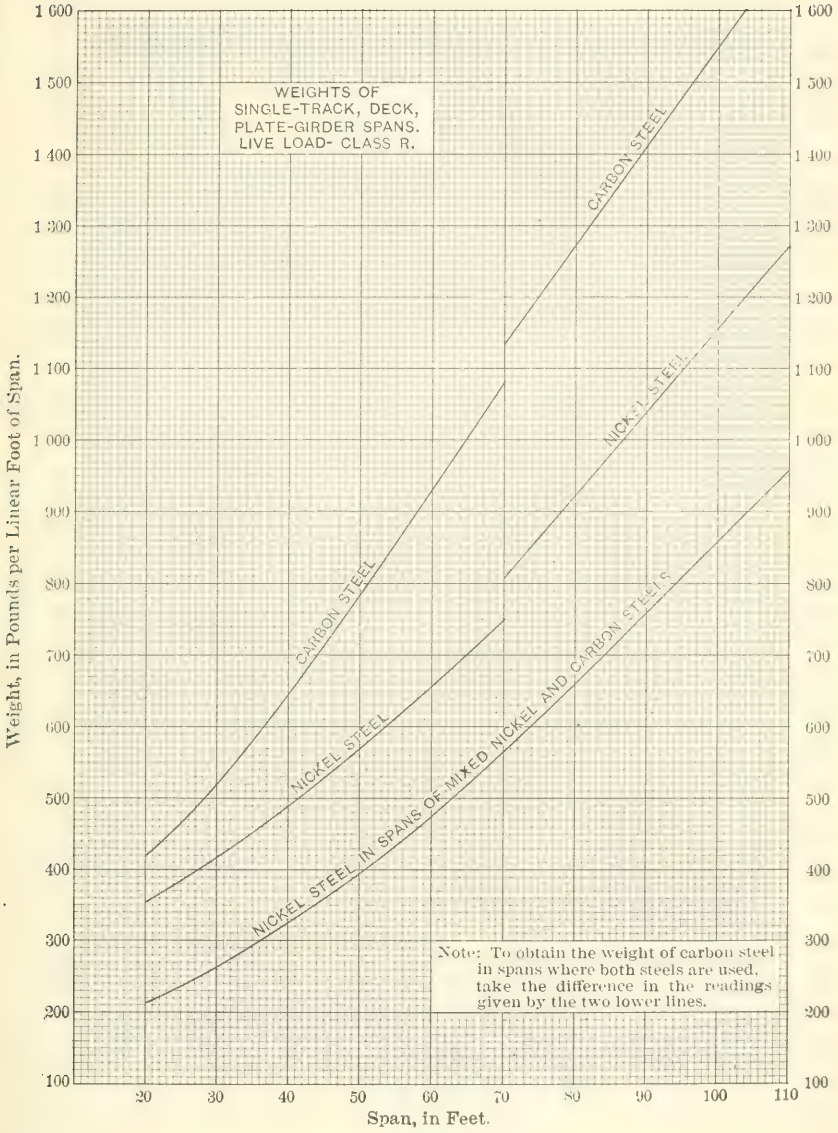

WEIGHTS OF
SINGLE-TRACK, DECK,
PLATE-GIRDER SPANS.
LIVE LOAD- CLASS R.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEEL
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
500
400
300
200
100
20
30
40
50
60
70
80
90
100
110
Span, in Feet.

FIG. 10.

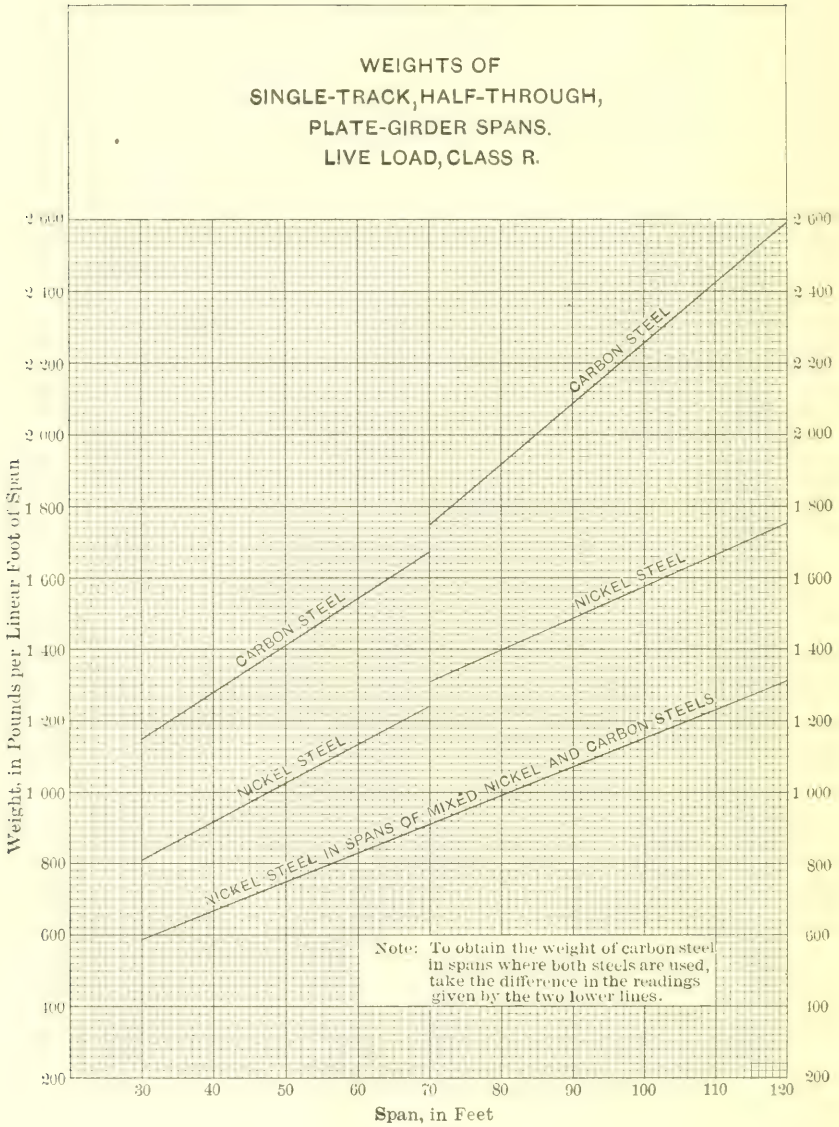

WEIGHTS OF
SINGLE-TRACK, HALF-THROUGH,
PLATE-GIRDER SPANS.
LIVE LOAD, CLASS R.
Weight, in Pounds per Linear Foot of Span
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
600
400
200
CARBON STEEL
NICKEL STEEL
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
30
40
50
60
70
80
90
100
110
120
Span, in Feet

FIG. 11.

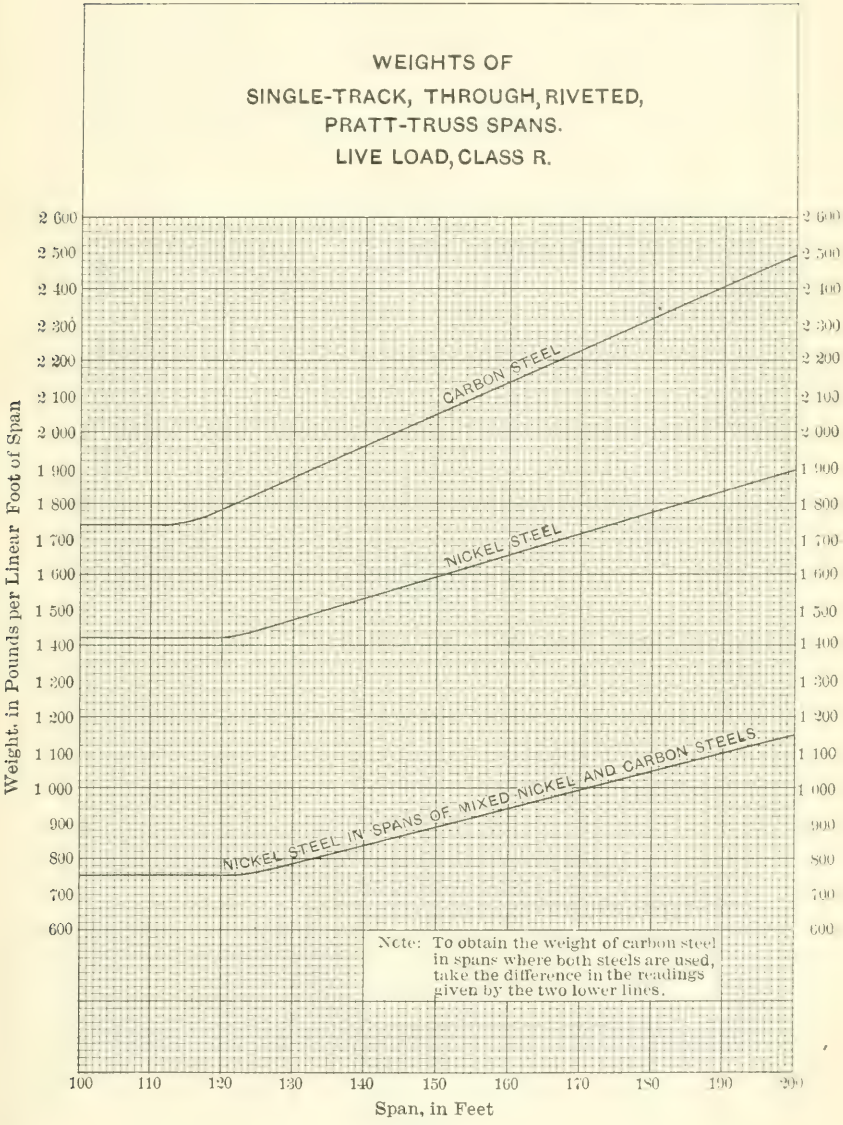

WEIGHTS OF
SINGLE-TRACK, THROUGH, RIVETED,
PRATT-TRUSS SPANS.
LIVE LOAD, CLASS R.
Weight, in Pounds per Lineal Foot of Span
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS.
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
100
110
120
130
140
150
160
170
180
190
200
Span, in Feet

FIG. 12.

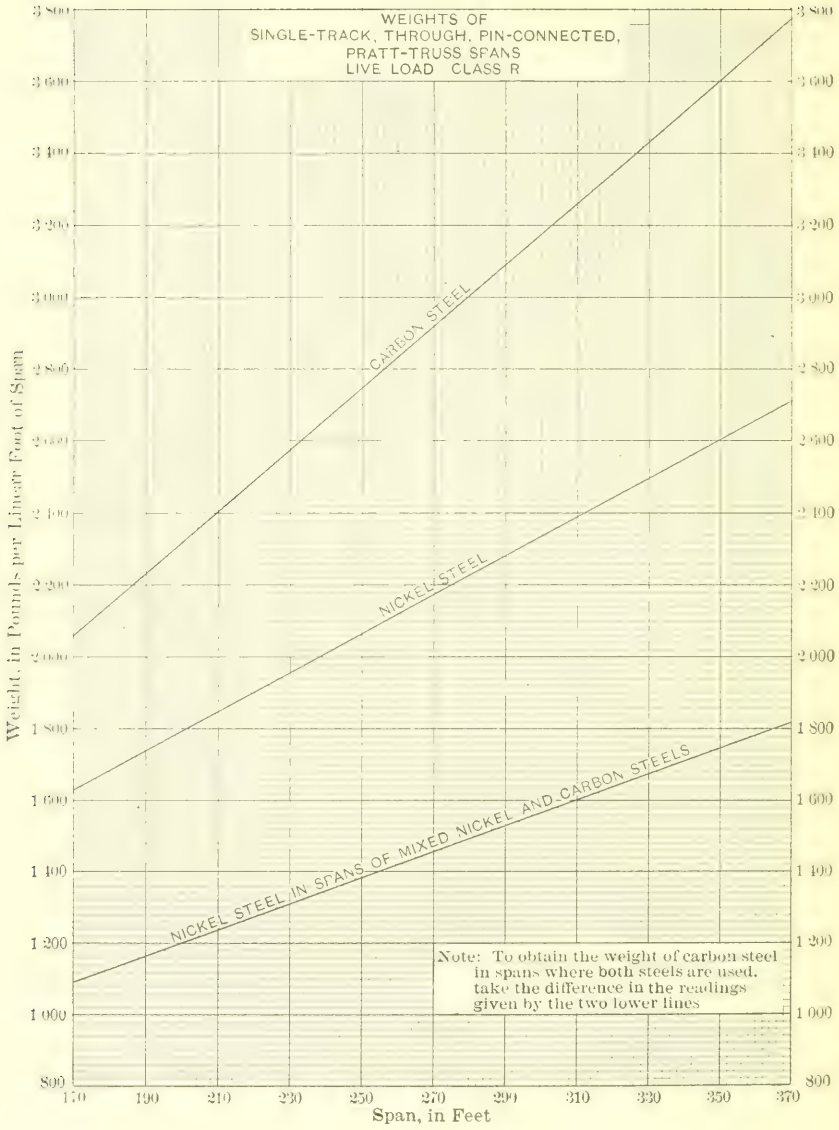

WEIGHTS OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS
LIVE LOAD CLASS R
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Span
Span, in Feet
3 800
3 600
3 400
3 200
3 000
2 800
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
170
190
210
230
250
270
290
310
330
350
370

FIG. 13.

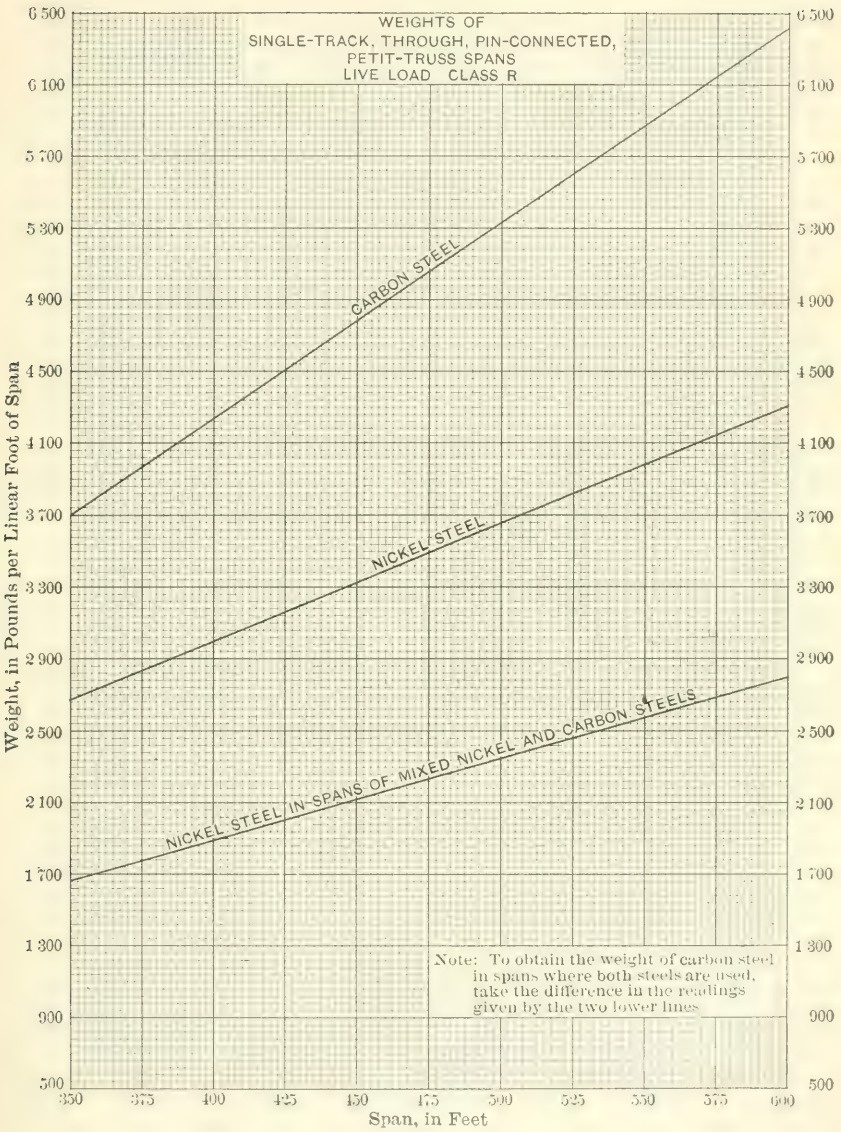

WEIGHTS OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS
LIVE LOAD CLASS R
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Span
6 500
6 100
5 700
5 300
4 900
4 500
4 100
3 700
3 300
2 900
2 500
2 100
1 700
1 300
900
500
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 14.

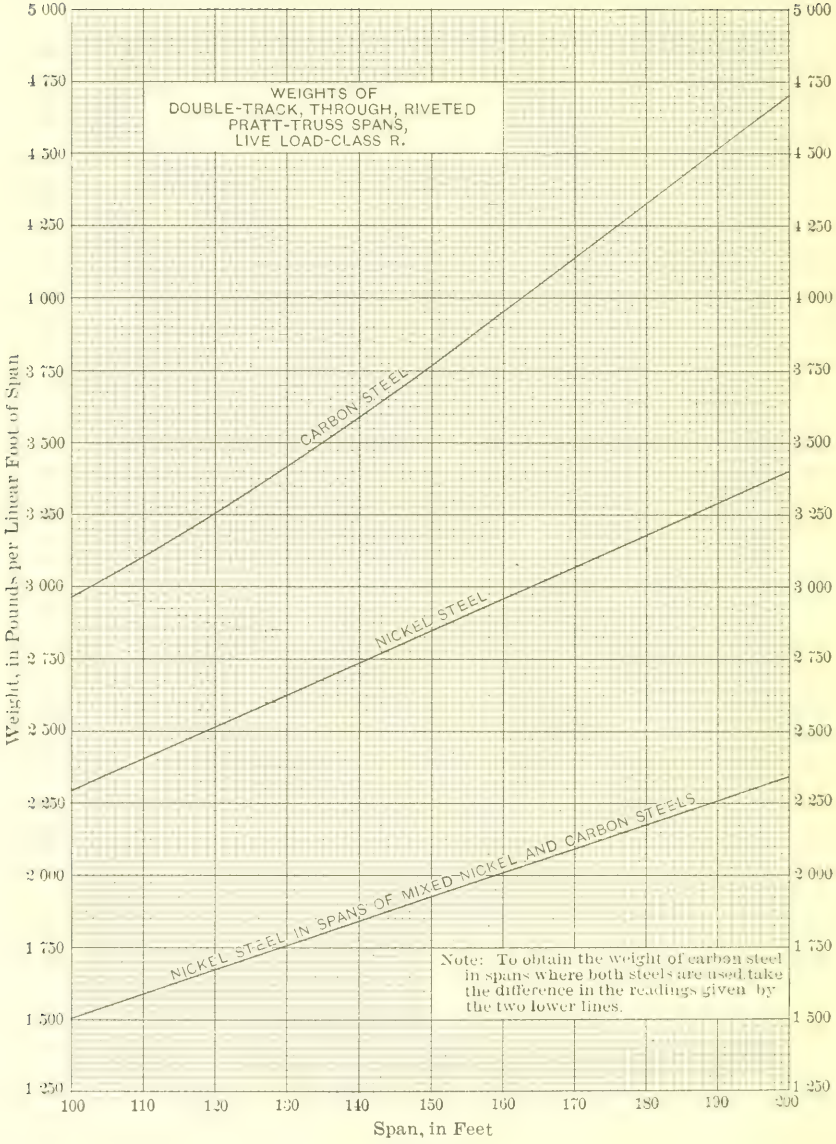

WEIGHTS OF
DOUBLE-TRACK, THROUGH, RIVETED
PRATT-TRUSS SPANS,
LIVE LOAD-CLASS R.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span
5 000
4 750
4 500
4 250
4 000
3 750
3 500
3 250
3 000
2 750
2 500
2 250
2 000
1 750
1 500
1 250
100
110
120
130
140
150
160
170
180
190
200
Span, in Feet

FIG. 15.

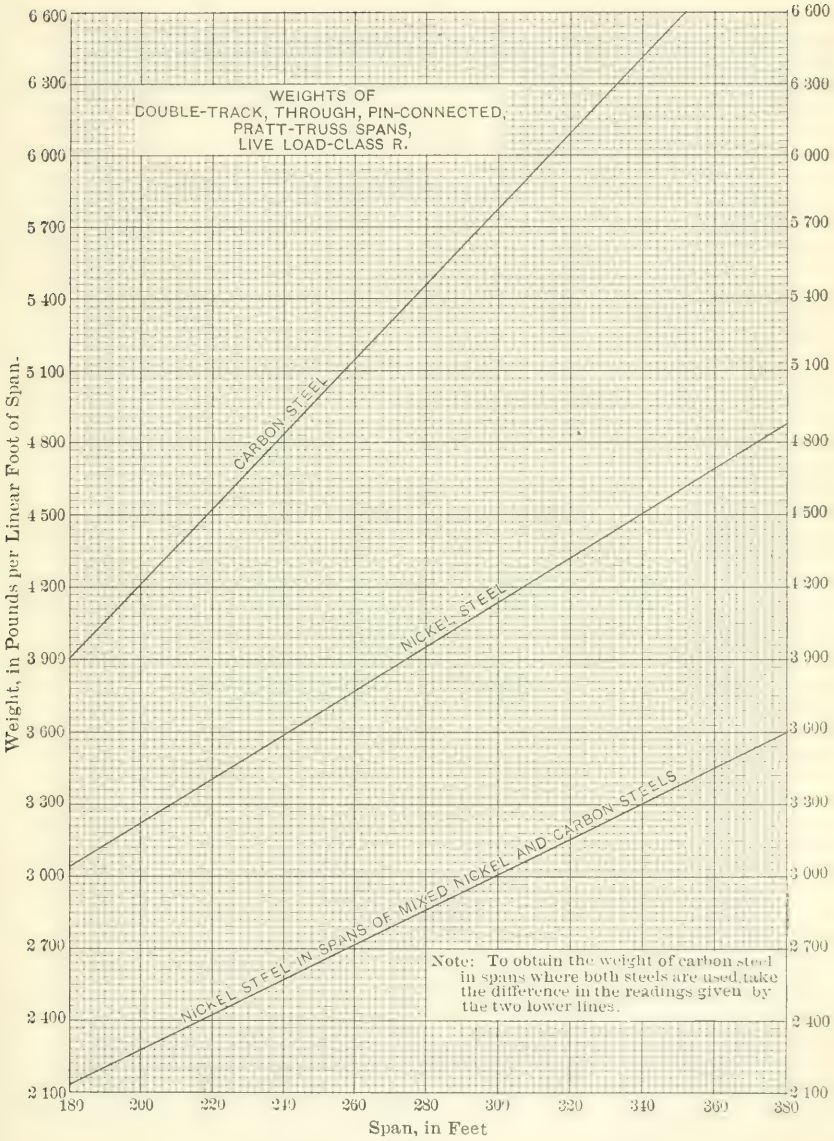

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS,
LIVE LOAD-CLASS R.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
6 600
6 300
6 000
5 700
5 400
5 100
4 800
4 500
4 200
3 900
3 600
3 300
3 000
2 700
2 400
2 100
180
200
220
240
260
280
300
320
340
360
380
Span, in Feet

FIG. 16.

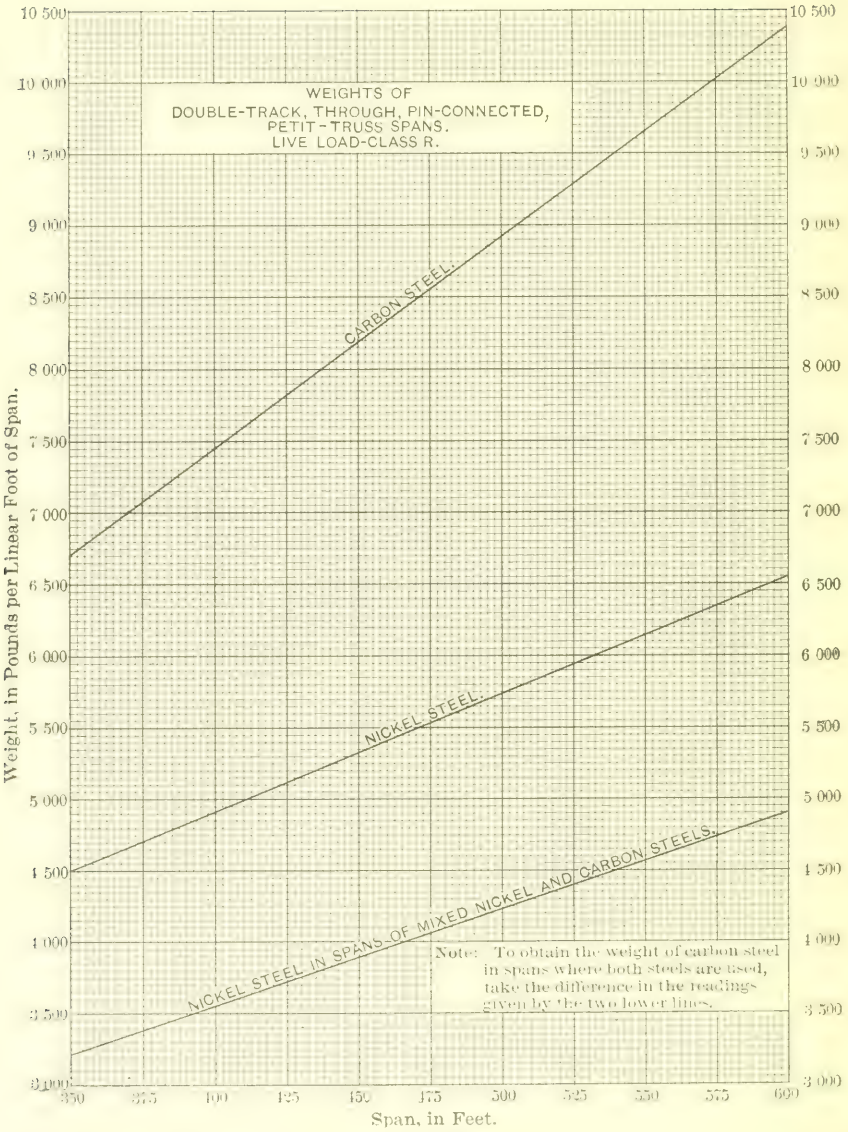

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS.
LIVE LOAD-CLASS R.
CARBON STEEL.
NICKEL STEEL.
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS.
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Span.
10 500
10 000
9 500
9 000
8 500
8 000
7 500
7 000
6 500
6 000
5 500
5 000
4 500
4 000
3 500
3 000
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet.

FIG. 17.

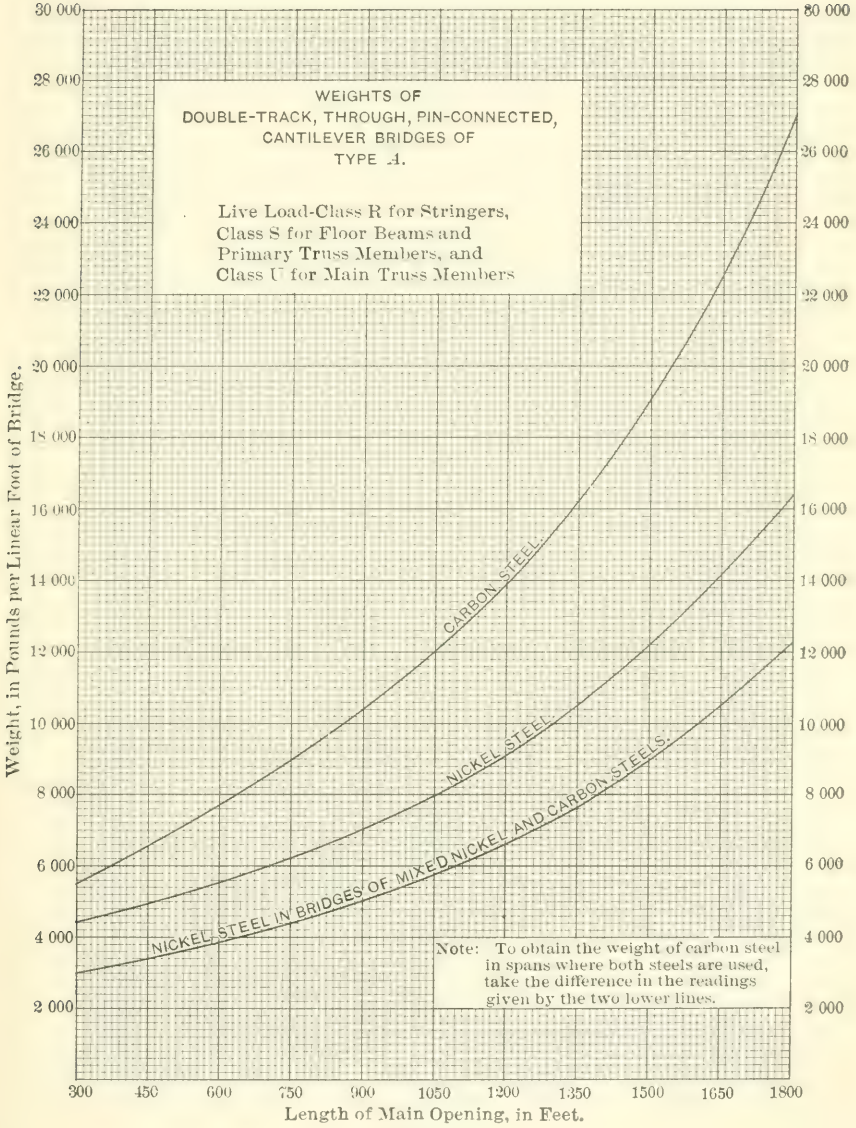

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE A.
Live Load-Class R for Stringers,
Class S for Floor Beams and
Primary Truss Members, and
Class U for Main Truss Members
CARBON STEEL.
NICKEL STEEL.
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS.
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Bridge.
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1050
1200
1350
1500
1650
1800
Length of Main Opening, in Feet.

FIG. 18

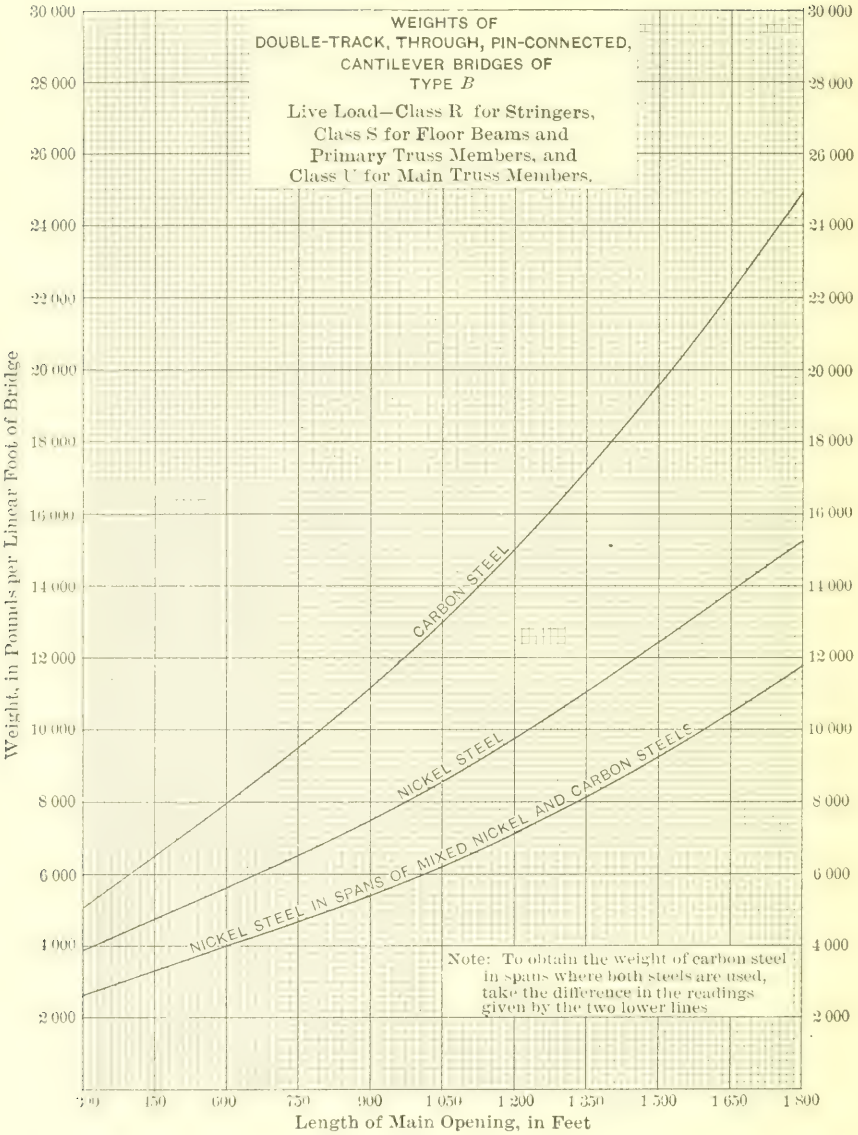

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE B
Live Load—Class R for Stringers,
Class S for Floor Beams and
Primary Truss Members, and
Class U for Main Truss Members.
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN SPANS OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
Weight, in Pounds per Linear Foot of Bridge
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 19.

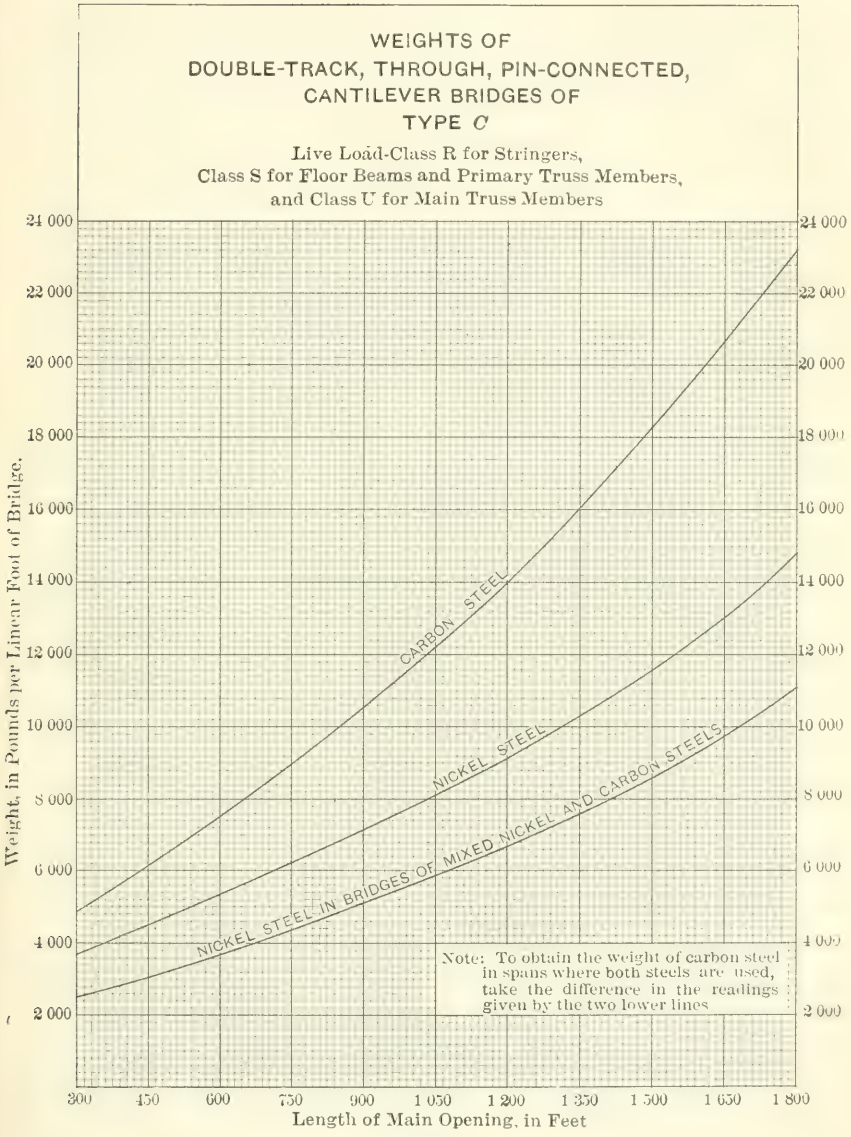

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE C
Live Load-Class R for Stringers,
Class S for Floor Beams and Primary Truss Members,
and Class U for Main Truss Members
Weight, in Pounds per Linear Foot of Bridge.
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 20.

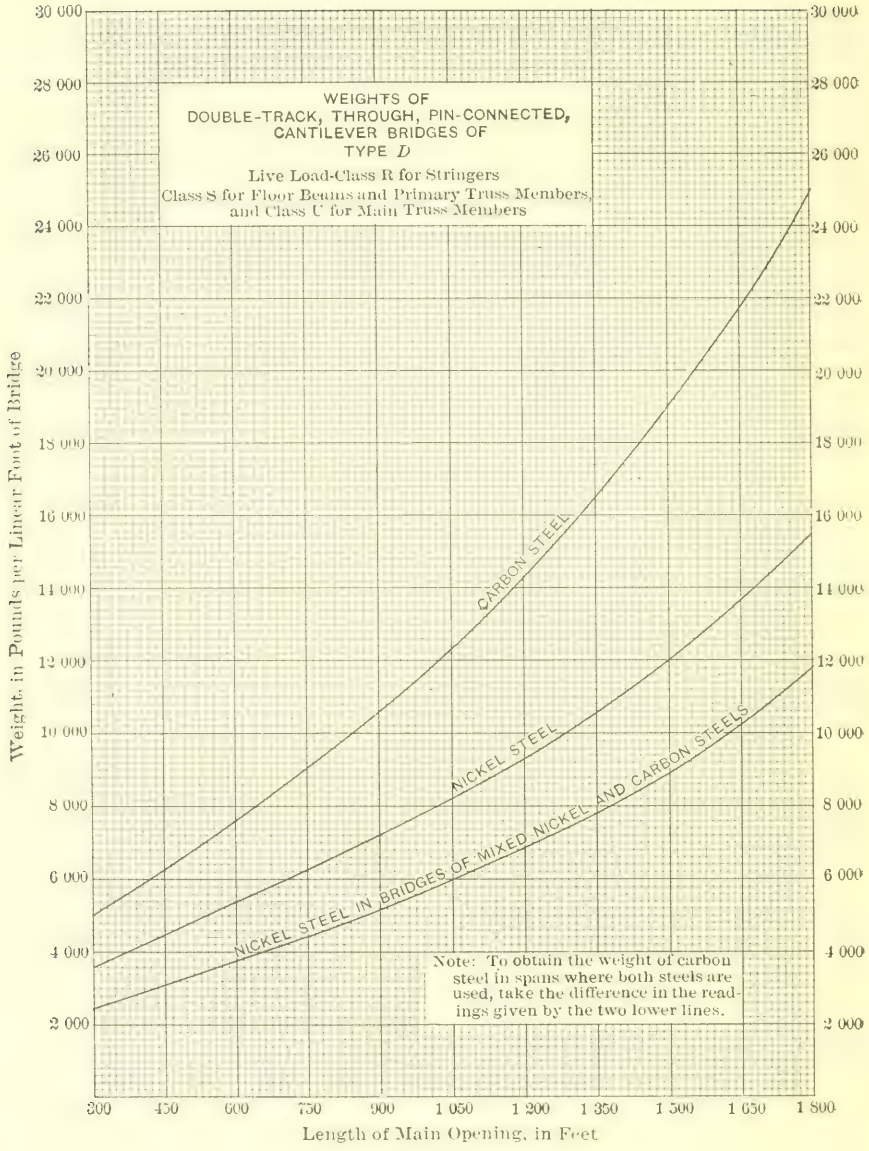

WEIGHTS OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
CANTILEVER BRIDGES OF
TYPE D
Live Load-Class R for Stringers
Class S for Floor Beams and Primary Truss Members,
and Class U for Main Truss Members
CARBON STEEL
NICKEL STEEL
NICKEL STEEL IN BRIDGES OF MIXED NICKEL AND CARBON STEELS
Note: To obtain the weight of carbon steel in spans where both steels are used, take the difference in the readings given by the two lower lines.
Weight, in Pounds per Linear Foot of Bridge
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 21.

DOUBLE-TRACK RAILROAD CANTILEVER BRIDGES. TYPICAL LAY-OUTS.

Trusses are spaced to agree with requirements of "De Pontibus."
Loading is Class R for stringers, Class S for floor beams and primary truss members, and Class U for main truss members.
Suspended spans are $\frac{3}{8}$ of main opening and cantilever and anchor arms are each $\frac{5}{16}$ of main opening.
l equals sum of one suspended span and two cantilever arms.

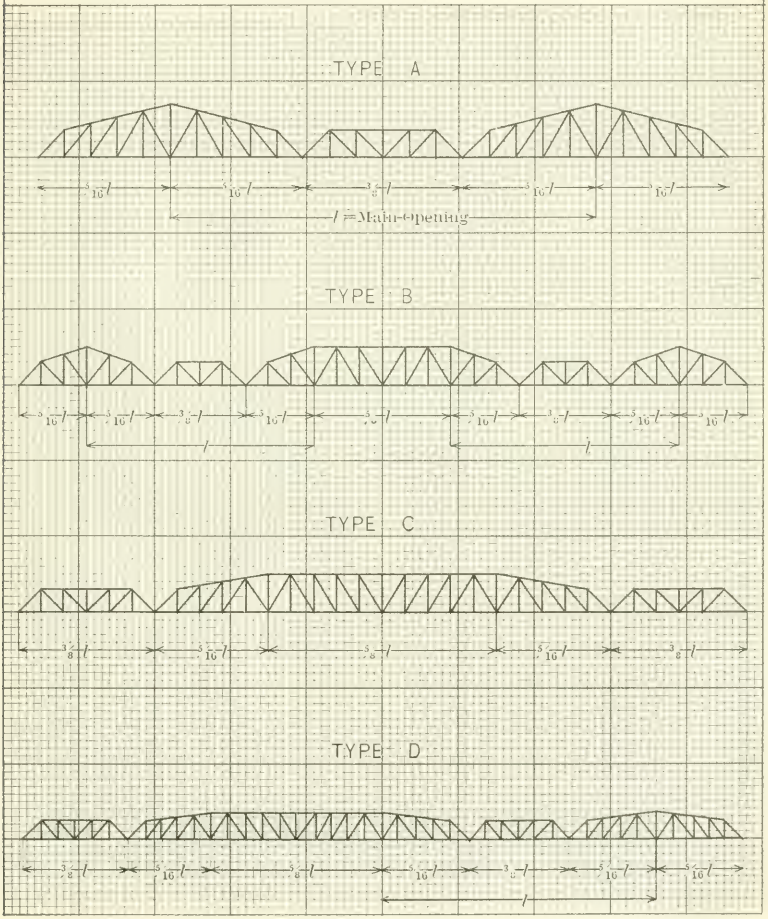
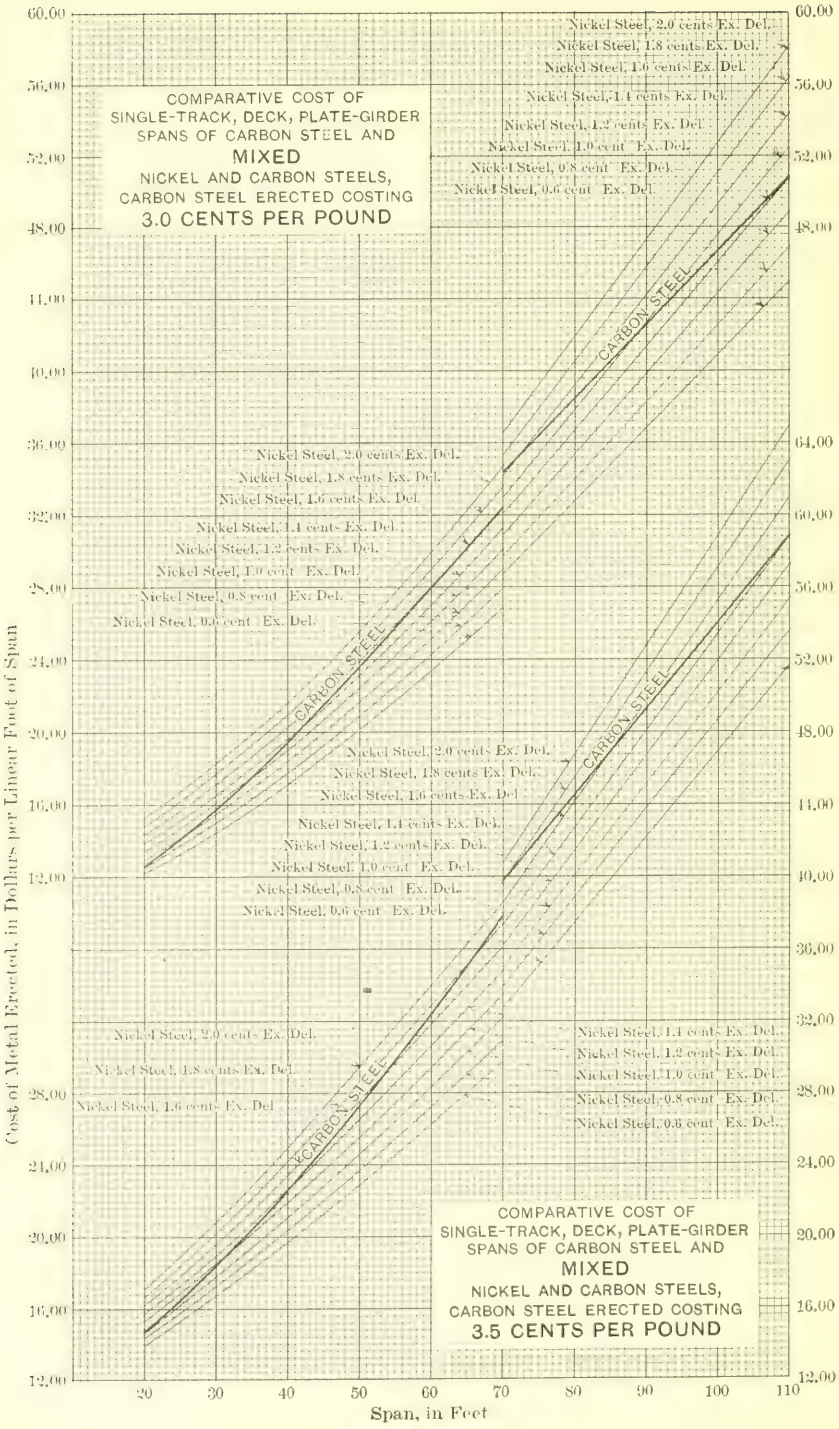


FIG. 22.



FIGS. 23 AND 24.

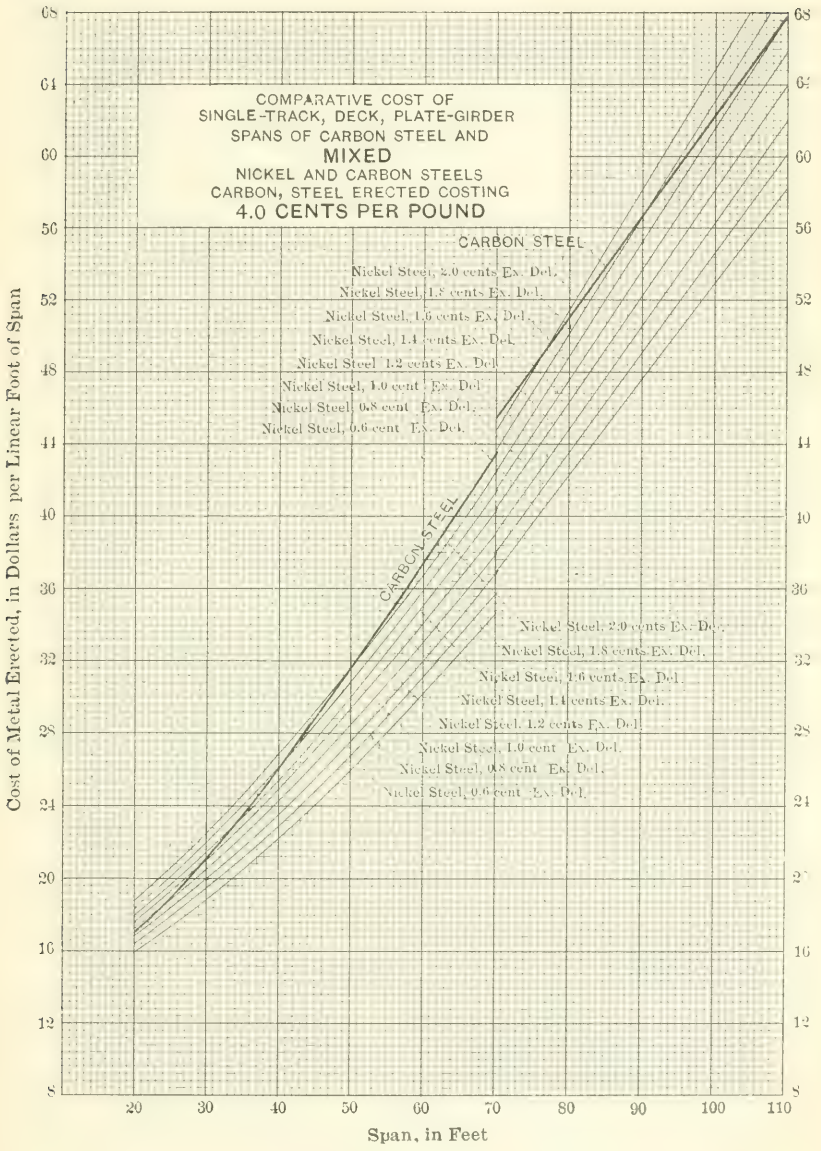
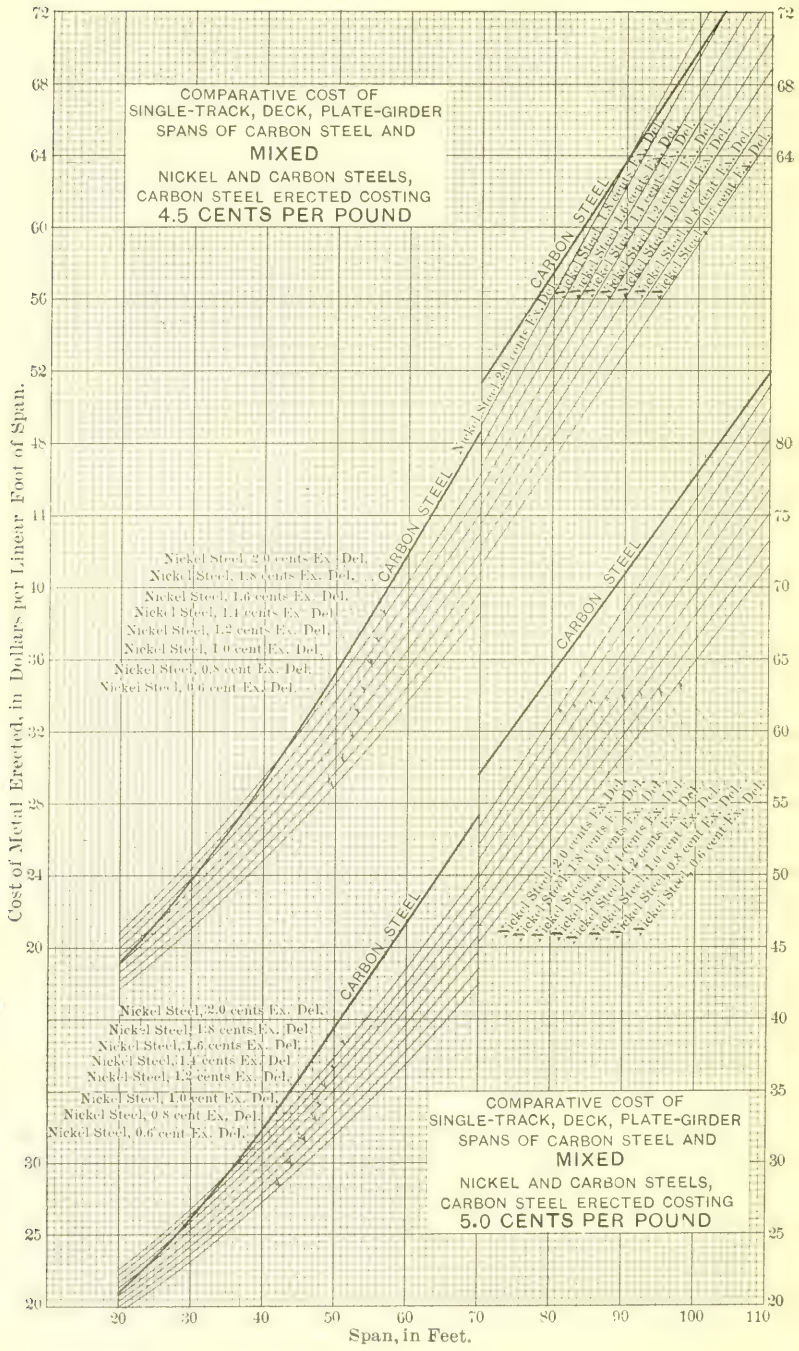

COMPARATIVE COST OF
SINGLE-TRACK, DECK, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS
CARBON, STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
20
30
40
50
60
70
80
90
100
110

FIG. 25.


COMPARATIVE COST OF
SINGLE-TRACK, DECK, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
COMPARATIVE COST OF
SINGLE-TRACK, DECK, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND

FIGS. 26 AND 27

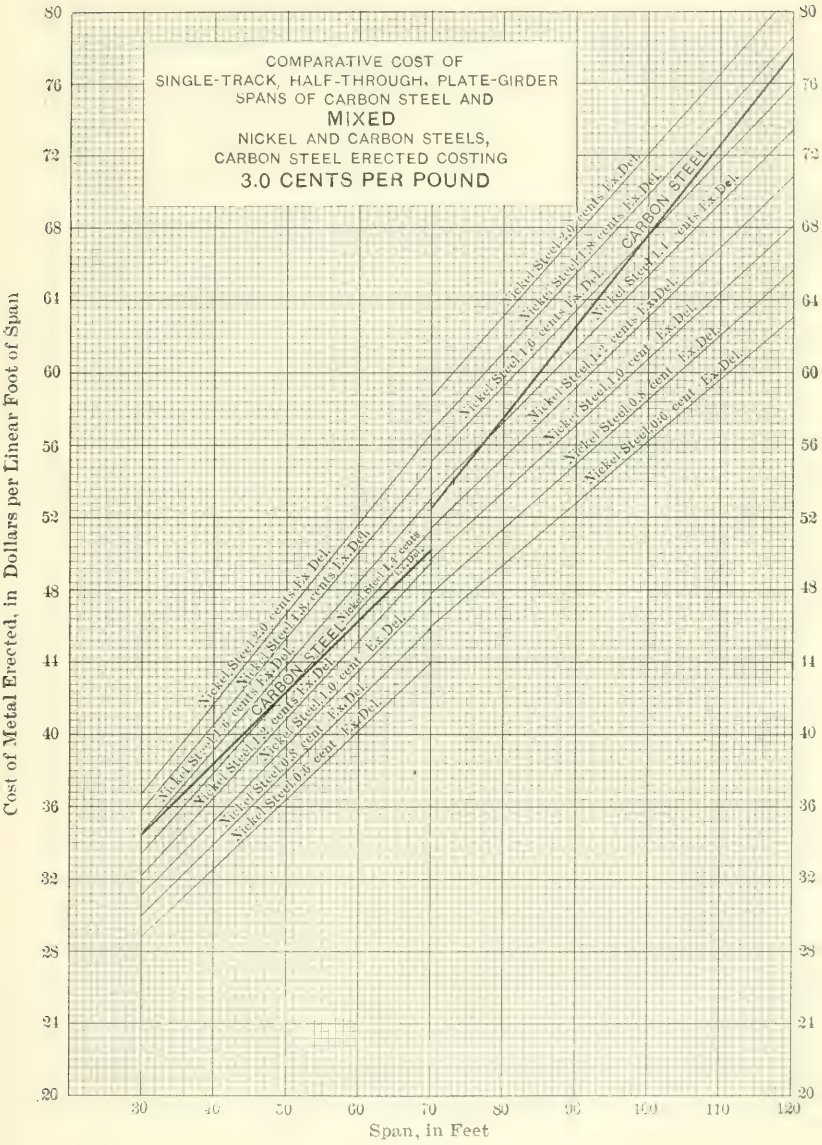

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.0 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
80
76
72
68
64
60
56
52
48
44
40
36
32
28
24
20
30
40
50
60
70
80
90
100
110
120
CARBON STEEL
Nickel Steel 2.0 cents Ex Del.
Nickel Steel 1.8 cents Ex Del.
Nickel Steel 1.6 cents Ex Del.
Nickel Steel 1.4 cents Ex Del.
Nickel Steel 1.2 cents Ex Del.
Nickel Steel 1.0 cent Ex Del.
Nickel Steel 0.8 cent Ex Del.
Nickel Steel 0.6 cent Ex Del.
Nickel Steel 2.0 cents Ex Del.
Nickel Steel 1.8 cents Ex Del.
Nickel Steel 1.6 cents Ex Del.
Nickel Steel 1.4 cents Ex Del.
Nickel Steel 1.2 cents Ex Del.
Nickel Steel 1.0 cent Ex Del.
Nickel Steel 0.8 cent Ex Del.
Nickel Steel 0.6 cent Ex Del.
CARBON STEEL
Nickel Steel 1.4 cents Ex Del.
Nickel Steel 1.2 cents Ex Del.
Nickel Steel 1.0 cent Ex Del.
Nickel Steel 0.8 cent Ex Del.
Nickel Steel 0.6 cent Ex Del.

FIG. 28.

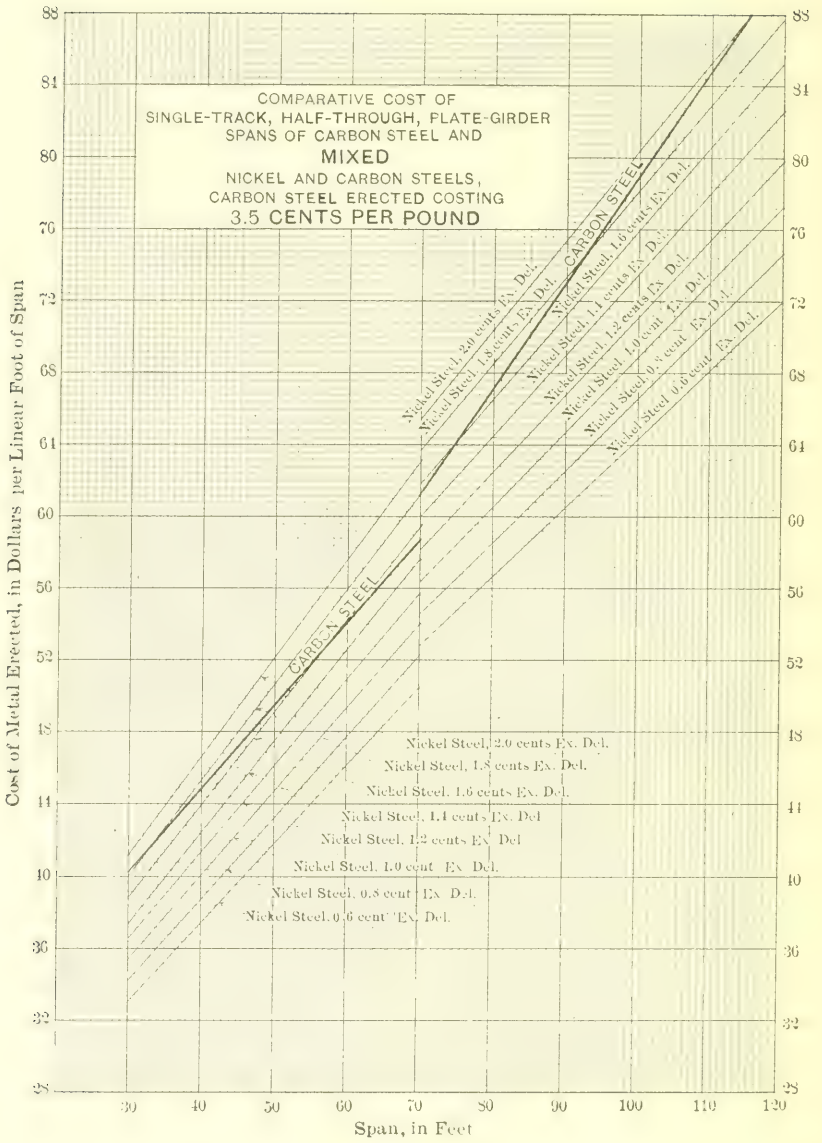

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cent Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.

FIG. 29.

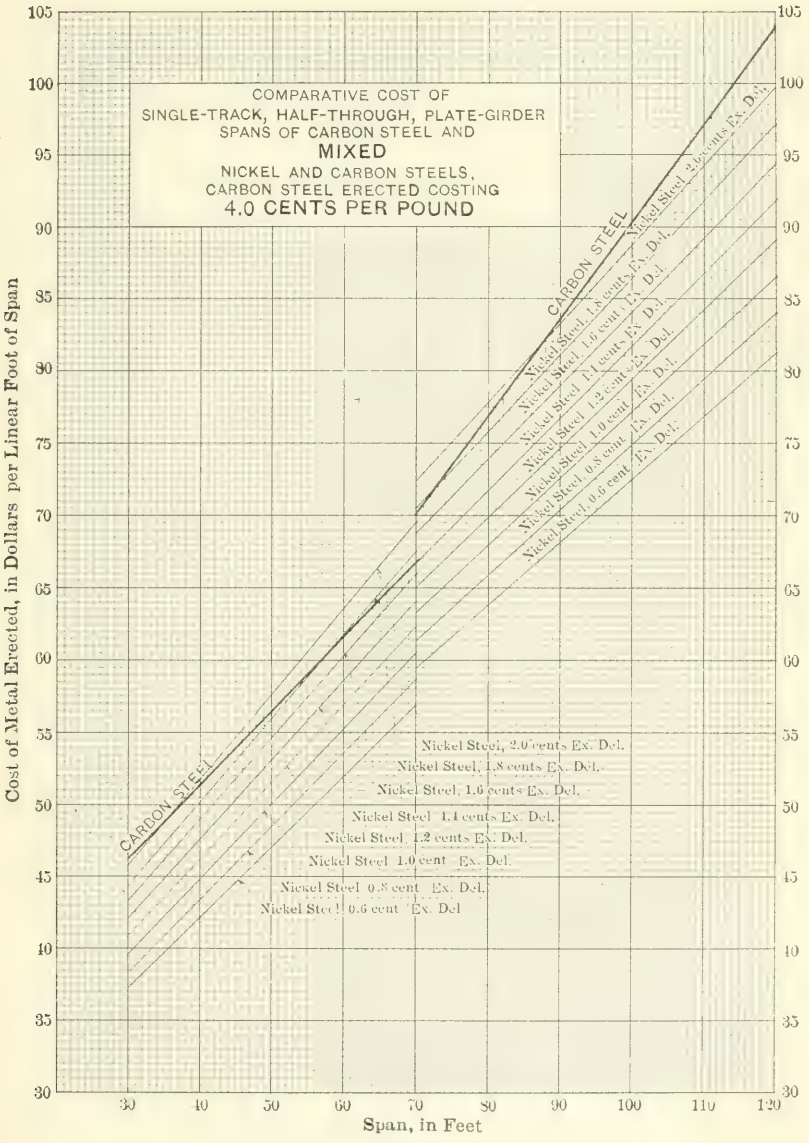

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel 1.0 cent Ex. Del.
Nickel Steel 0.8 cent Ex. Del.
Nickel Steel 0.6 cent Ex. Del

FIG. 30.

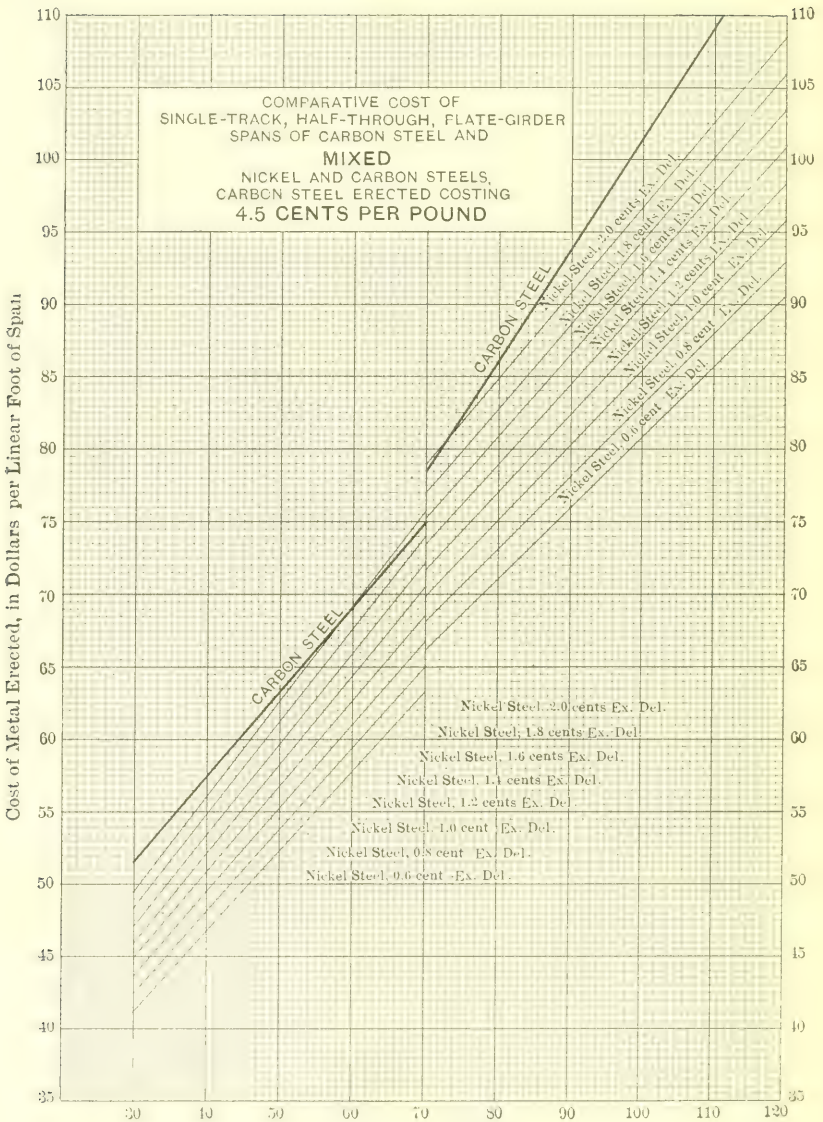

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
40
50
60
70
80
90
100
110
120

FIG. 31.

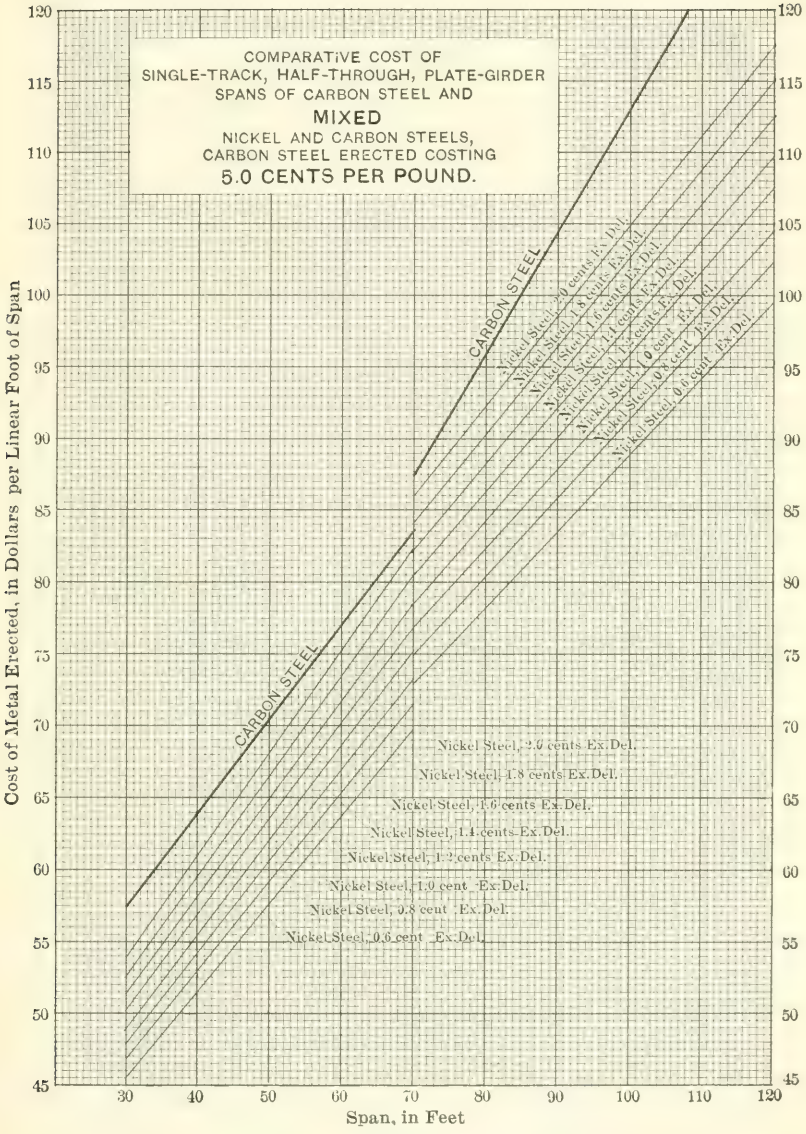
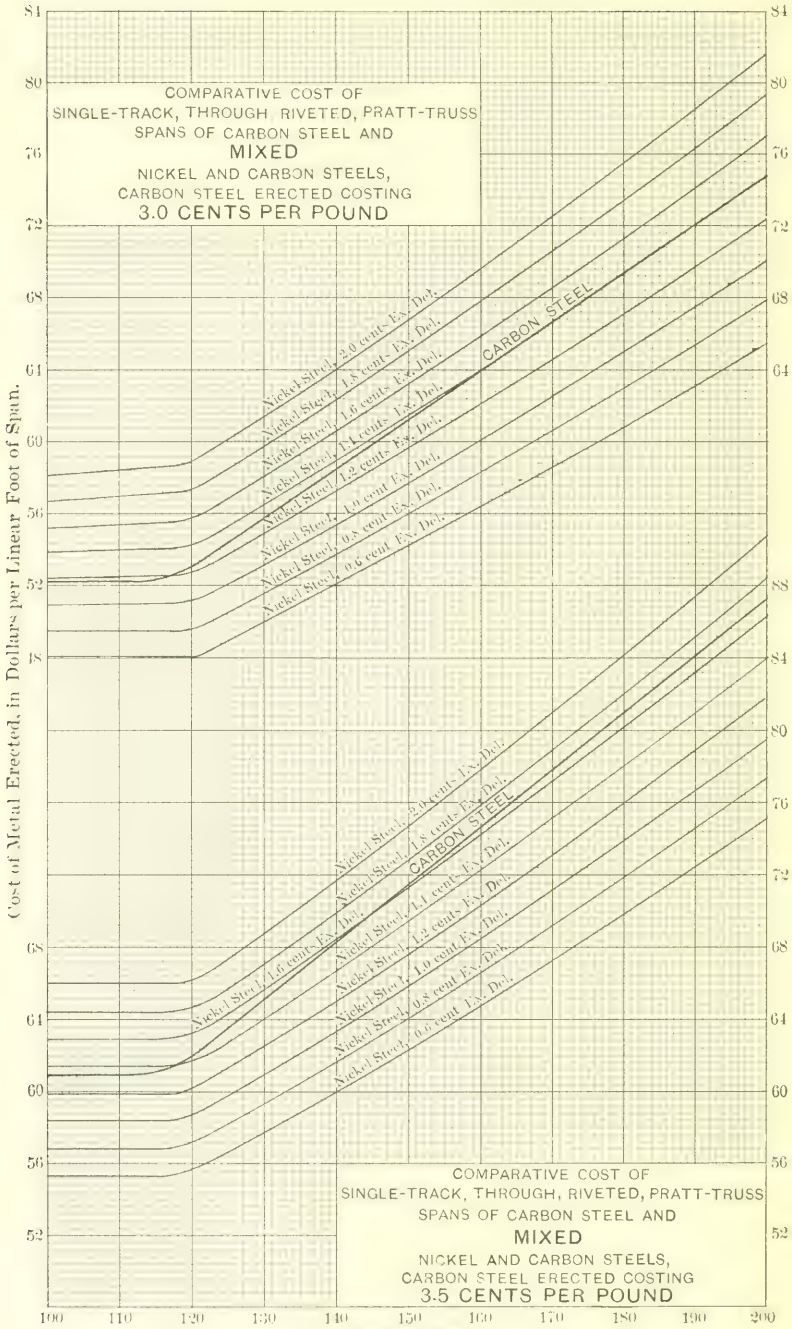
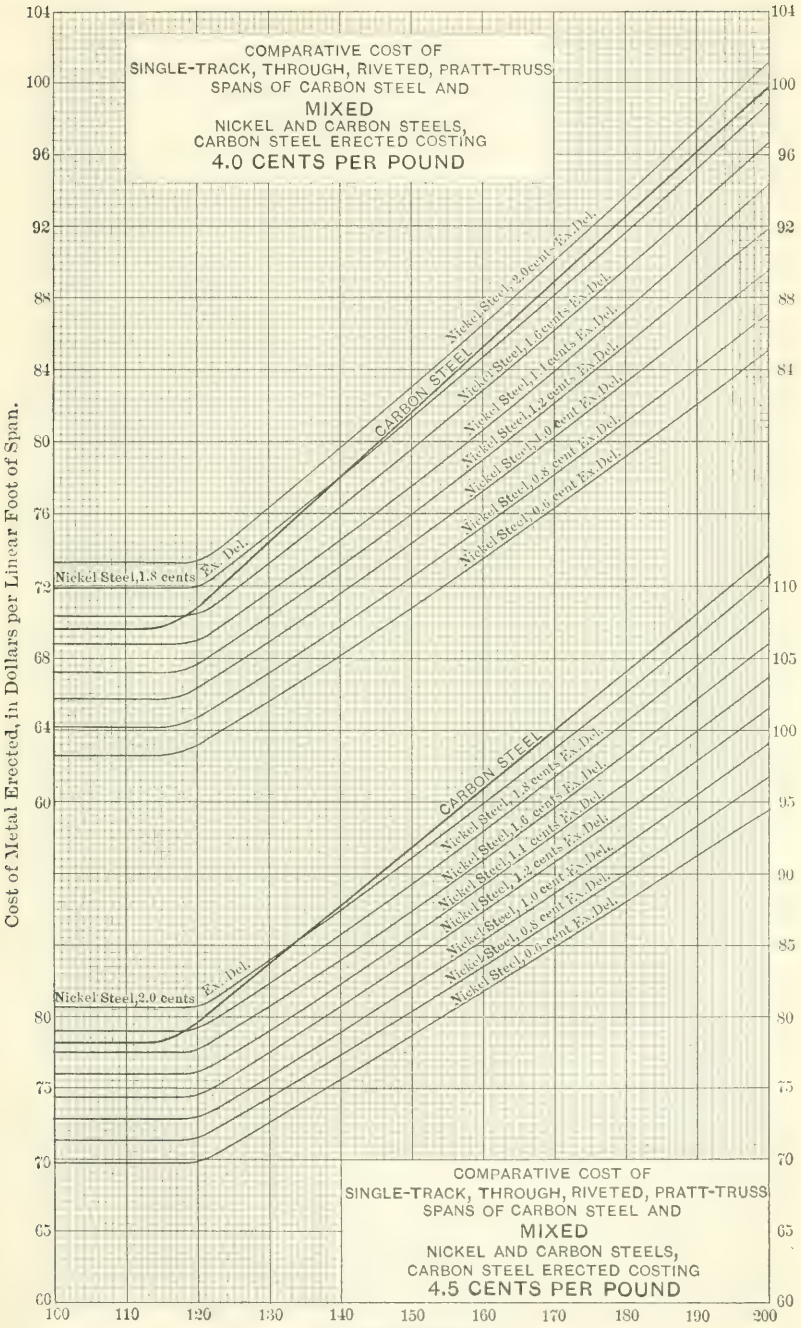

COMPARATIVE COST OF
SINGLE-TRACK, HALF-THROUGH, PLATE-GIRDER
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND.
CARBON STEEL
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
45
30
40
50
60
70
80
90
100
110
120

FIG. 32.



FIGS. 33 AND 34.


COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cent. Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cent. Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 2.0 cents Ex. Del.
CARBON STEEL
Nickel Steel, 1.8 cents, Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Nickel Steel, 2.0 cents Ex. Del.
COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet.
100 110 120 130 140 150 160 170 180 190 200
60 65 70 75 80 84 88 92 96 100 104
60 65 70 75 80 85 90 95 100 105 110

FIGS. 35 AND 36.

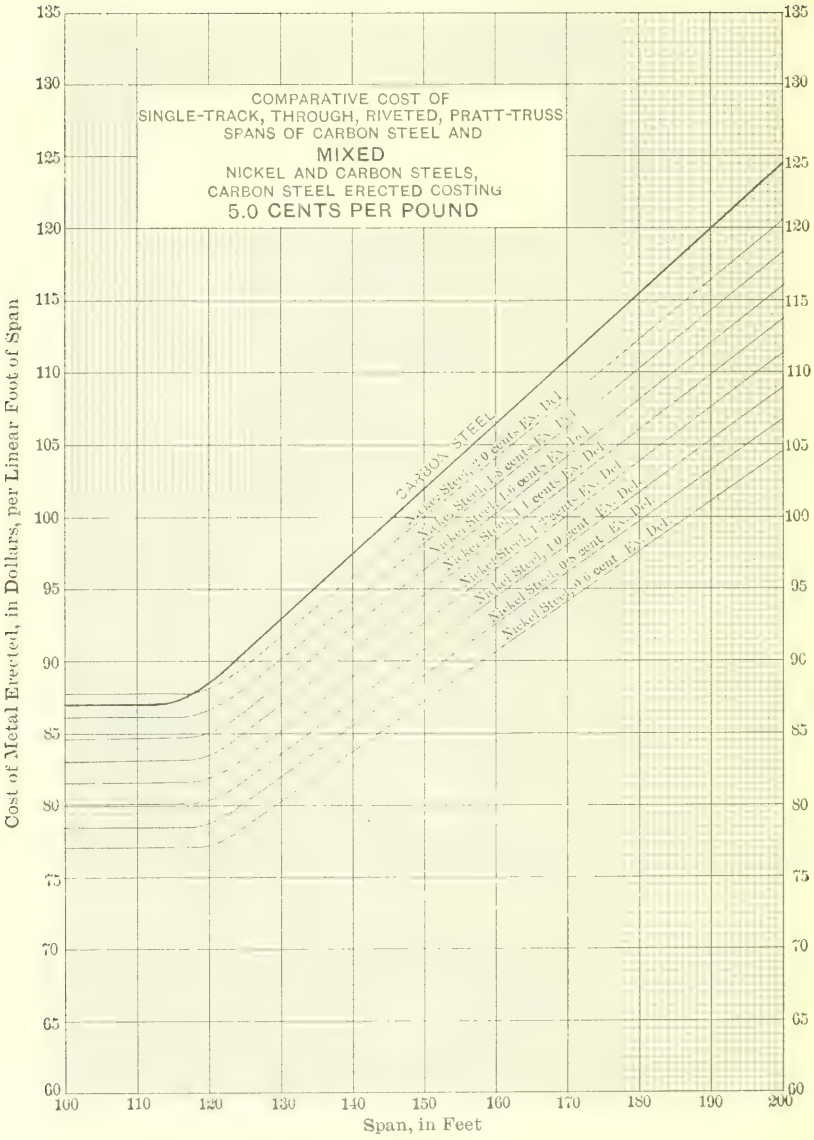
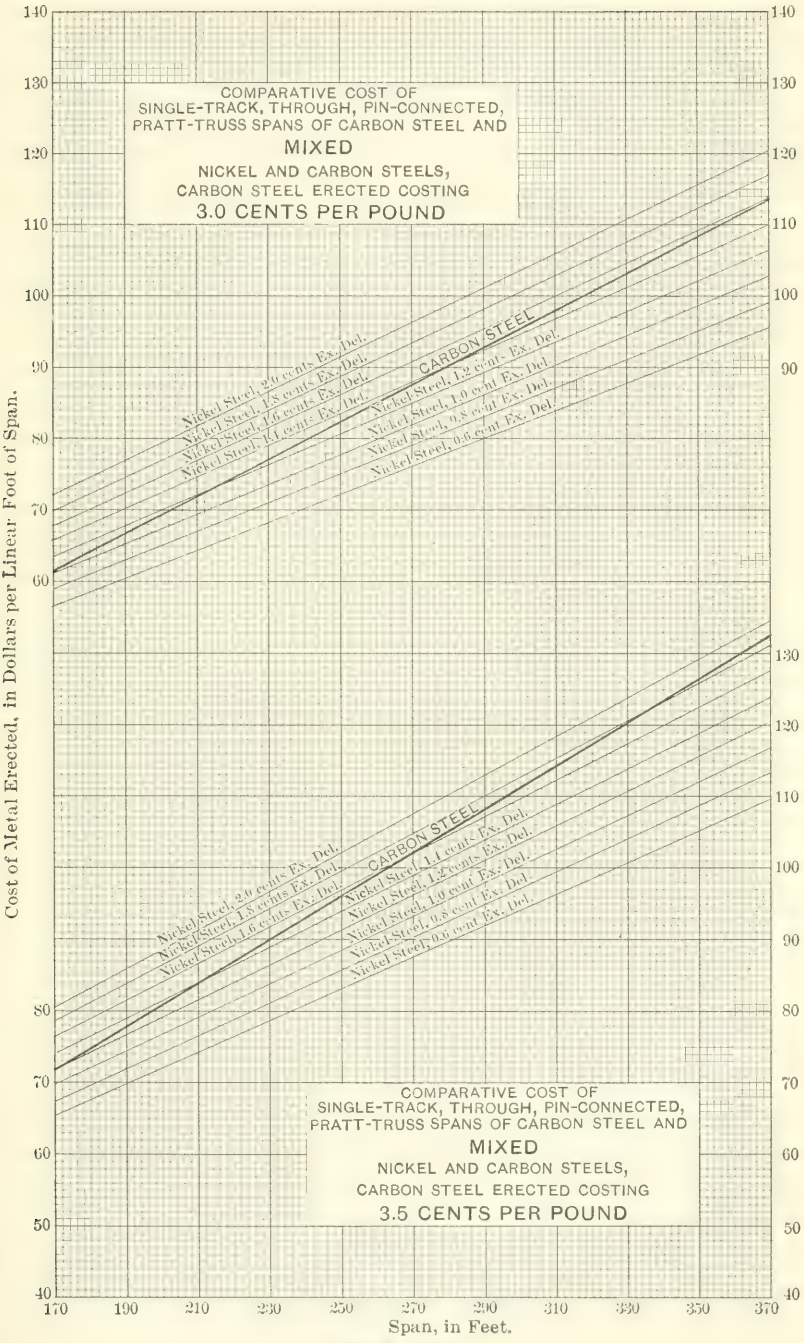
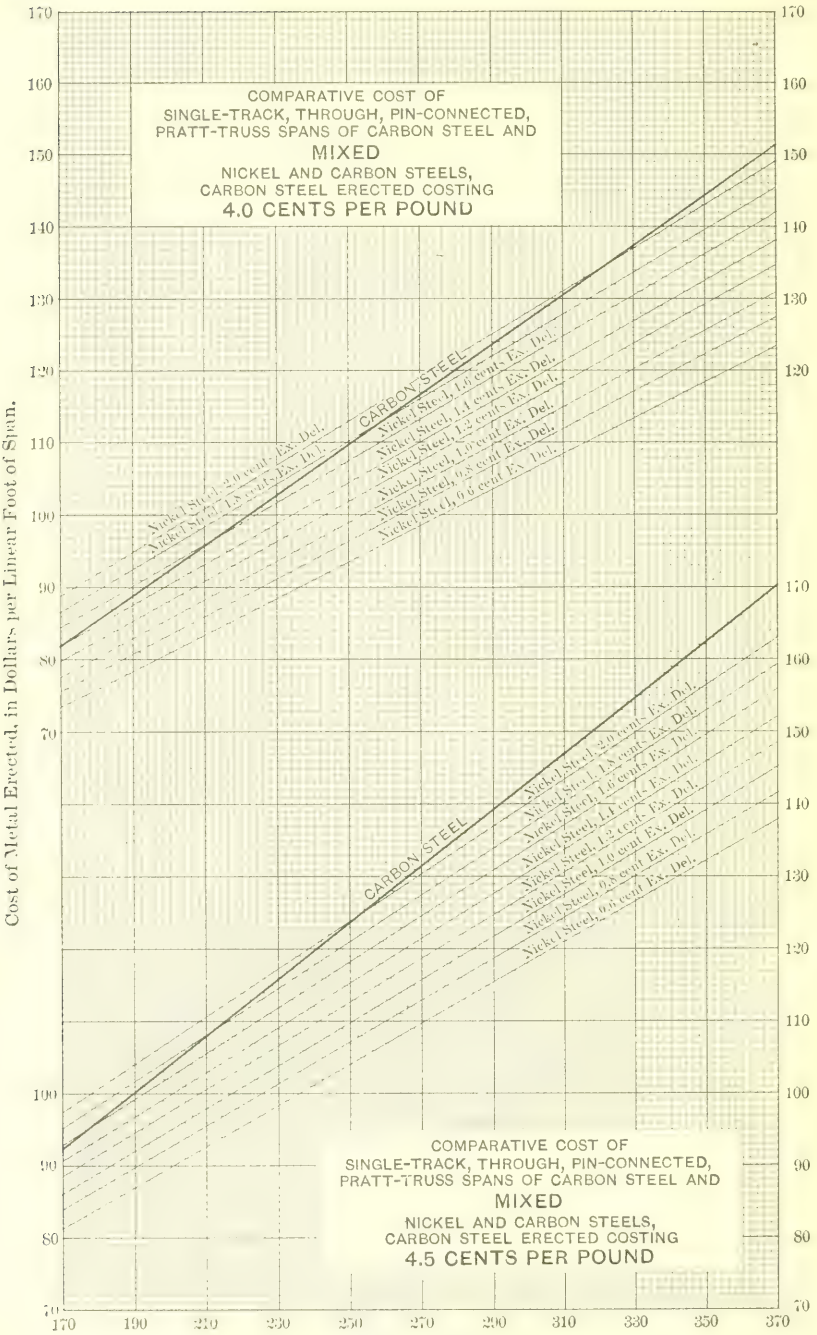

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cent. Ex. Del.
Nickel Steel, 1.5 cent. Ex. Del.
Nickel Steel, 1.0 cent. Ex. Del.
Nickel Steel, 1 cents. Ex. Del.
Nickel Steel, 1 cent. Ex. Del.
Nickel Steel, 1.0 cent. Ex. Del.
Nickel Steel, 0.5 cent. Ex. Del.
Nickel Steel, 0 cent. Ex. Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
135
130
125
120
115
110
105
100
95
90
85
80
75
70
65
60
100
110
120
130
140
150
160
170
180
190
200
Span, in Feet

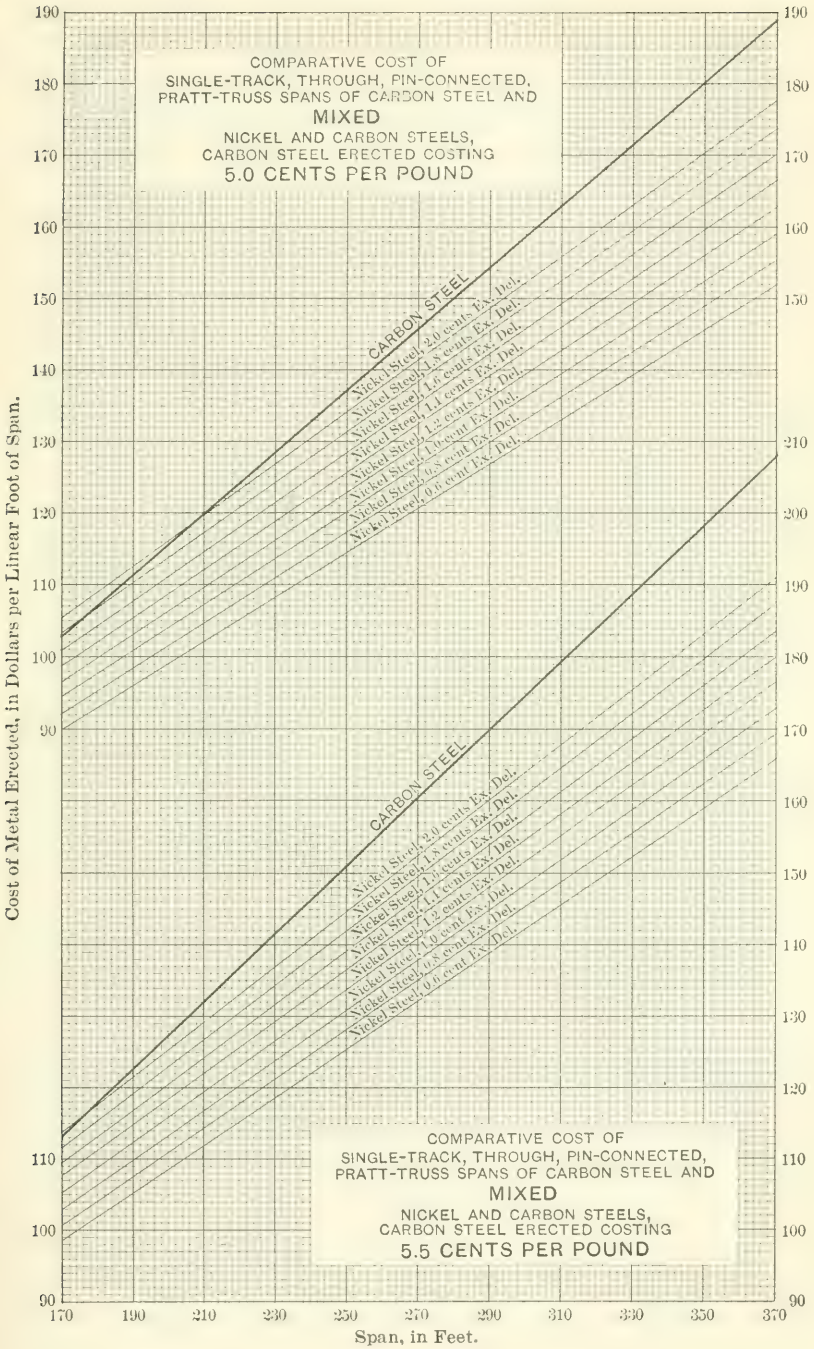
FIG. 37.



FIGS. 38 AND 39.


COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cent. Ex. Del.
Nickel Steel, 1.8 cent. Ex. Del.
Nickel Steel, 1.6 cent. Ex. Del.
Nickel Steel, 1.4 cent. Ex. Del.
Nickel Steel, 1.2 cent. Ex. Del.
Nickel Steel, 1.0 cent. Ex. Del.
Nickel Steel, 0.8 cent. Ex. Del.
Nickel Steel, 0.6 cent. Ex. Del.
COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet.

FIGS. 40 AND 41.



FIGS. 42 AND 43.

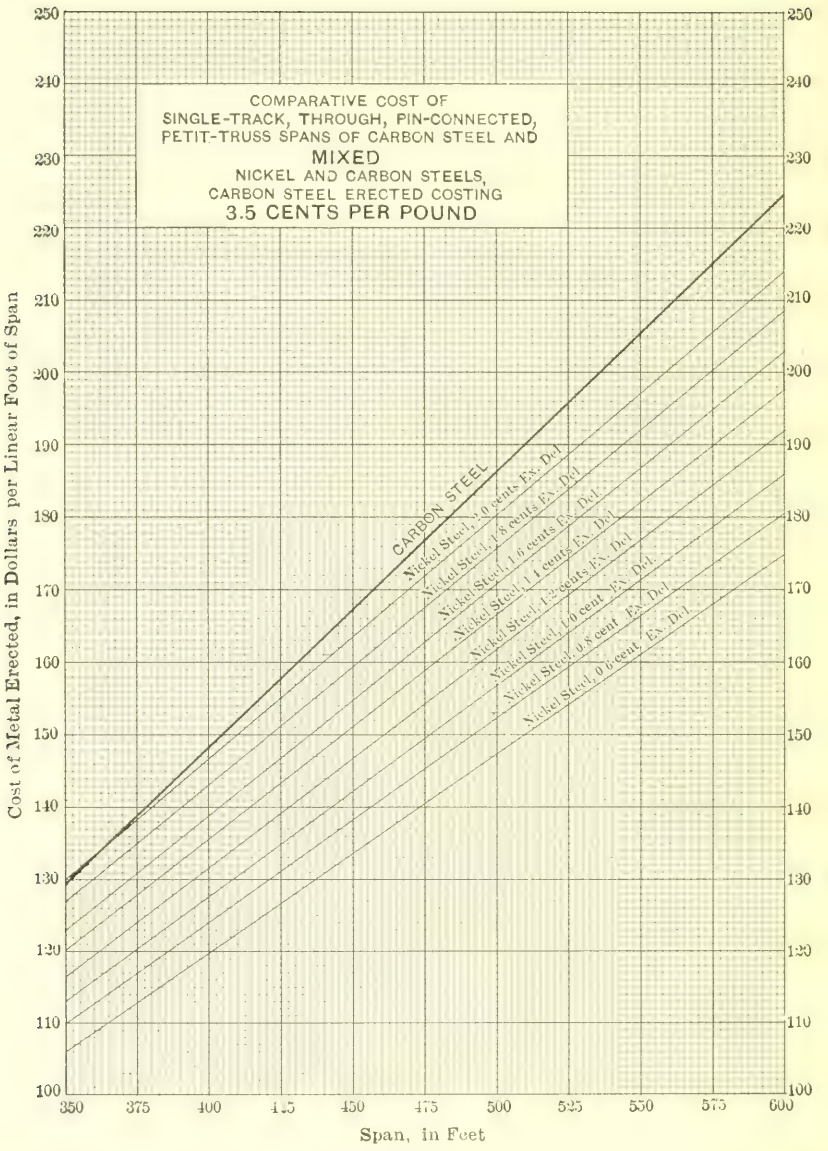

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
350
375
400
425
450
475
500
525
550
575
600

FIG. 44.

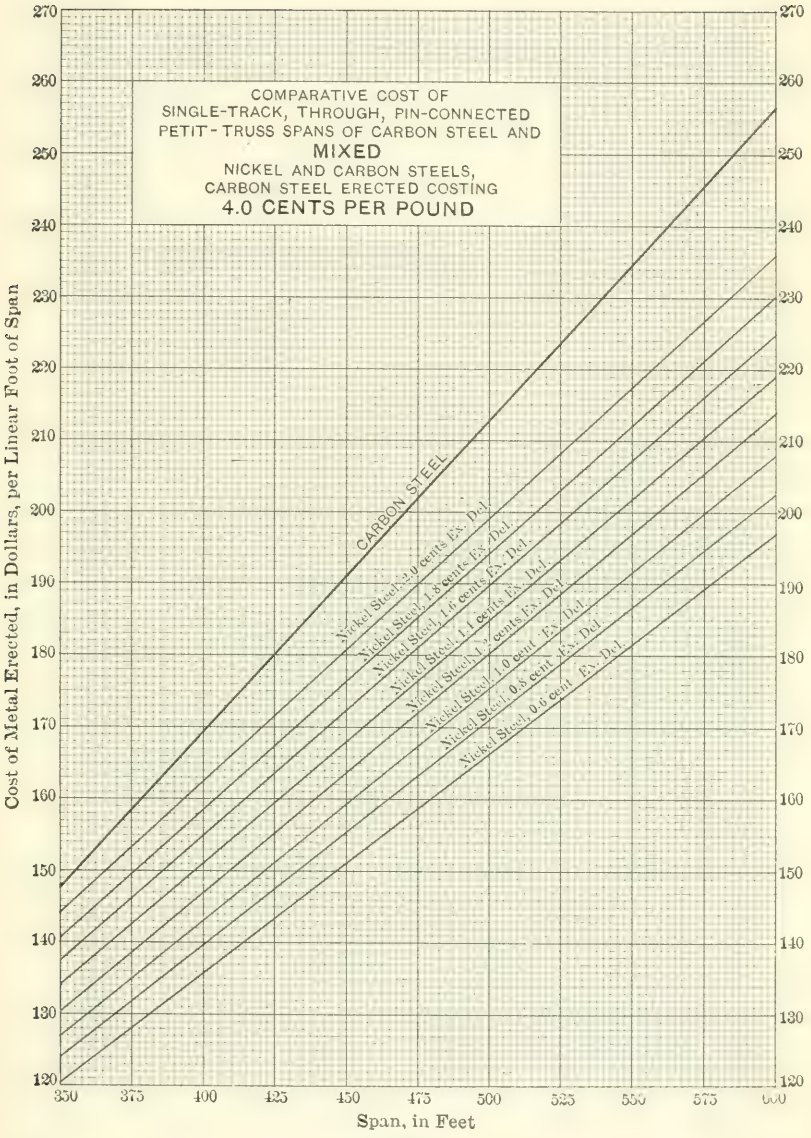
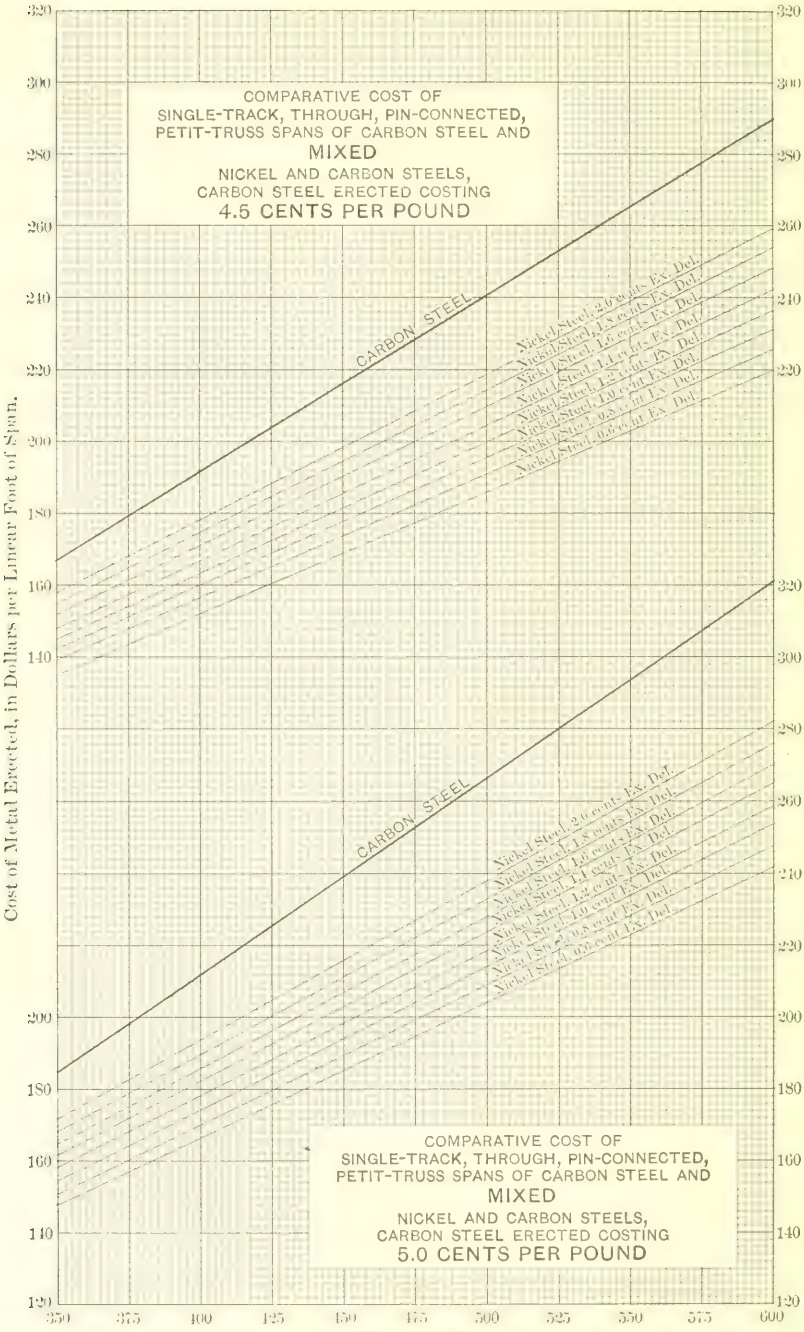

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
270
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 45.



FIGS. 46 AND 47.

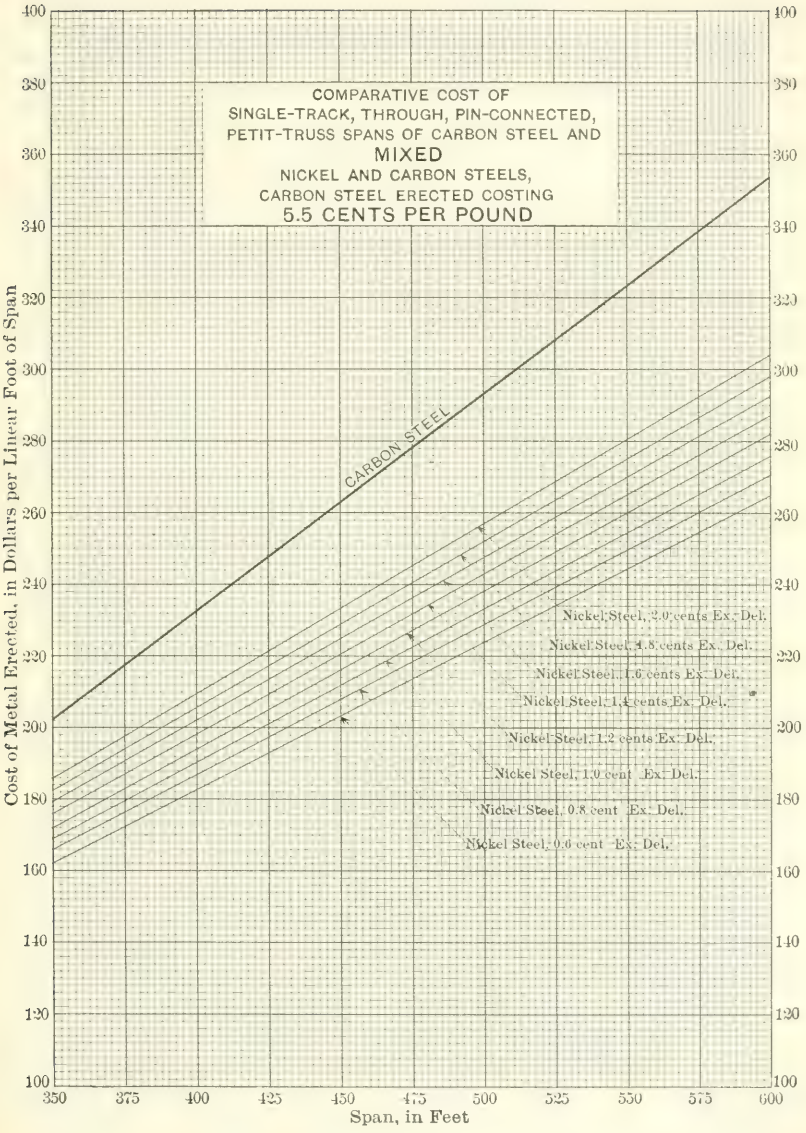
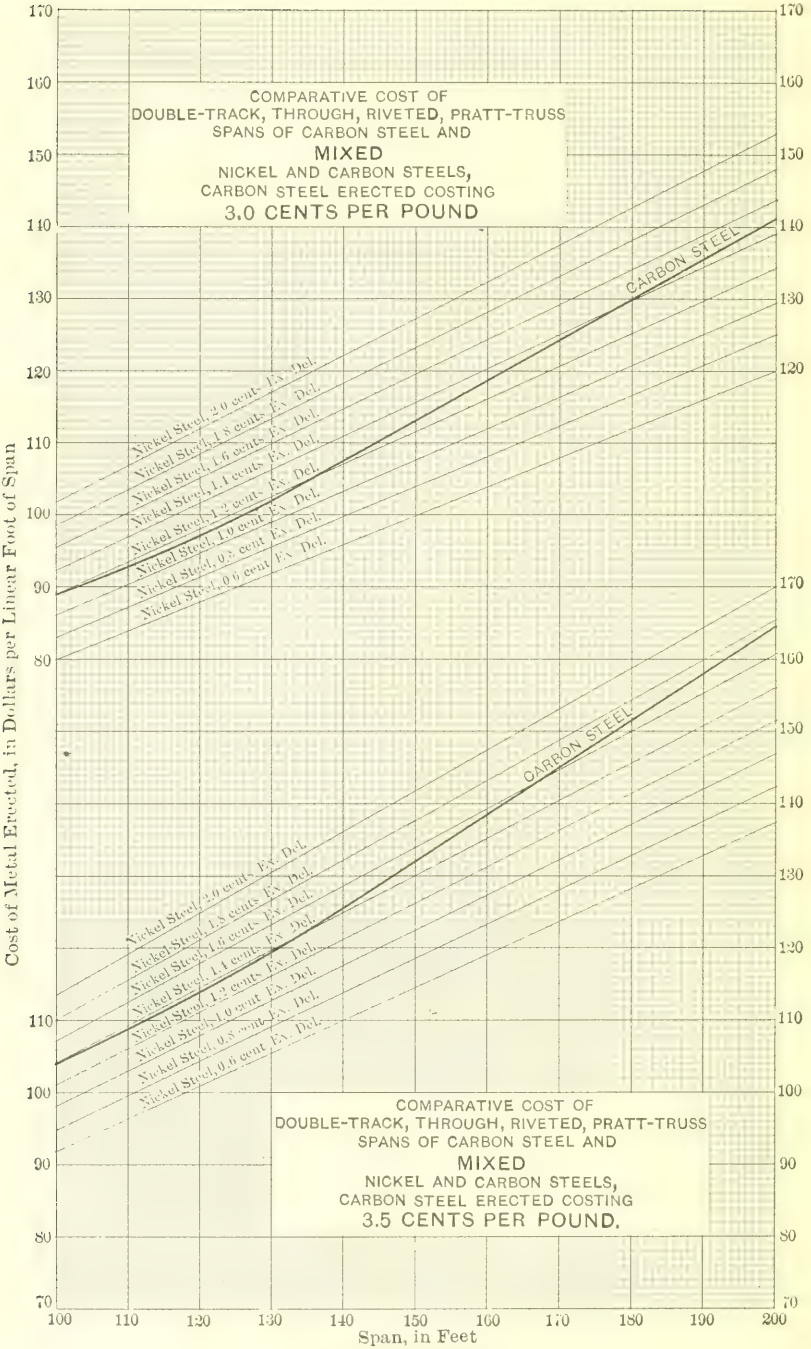
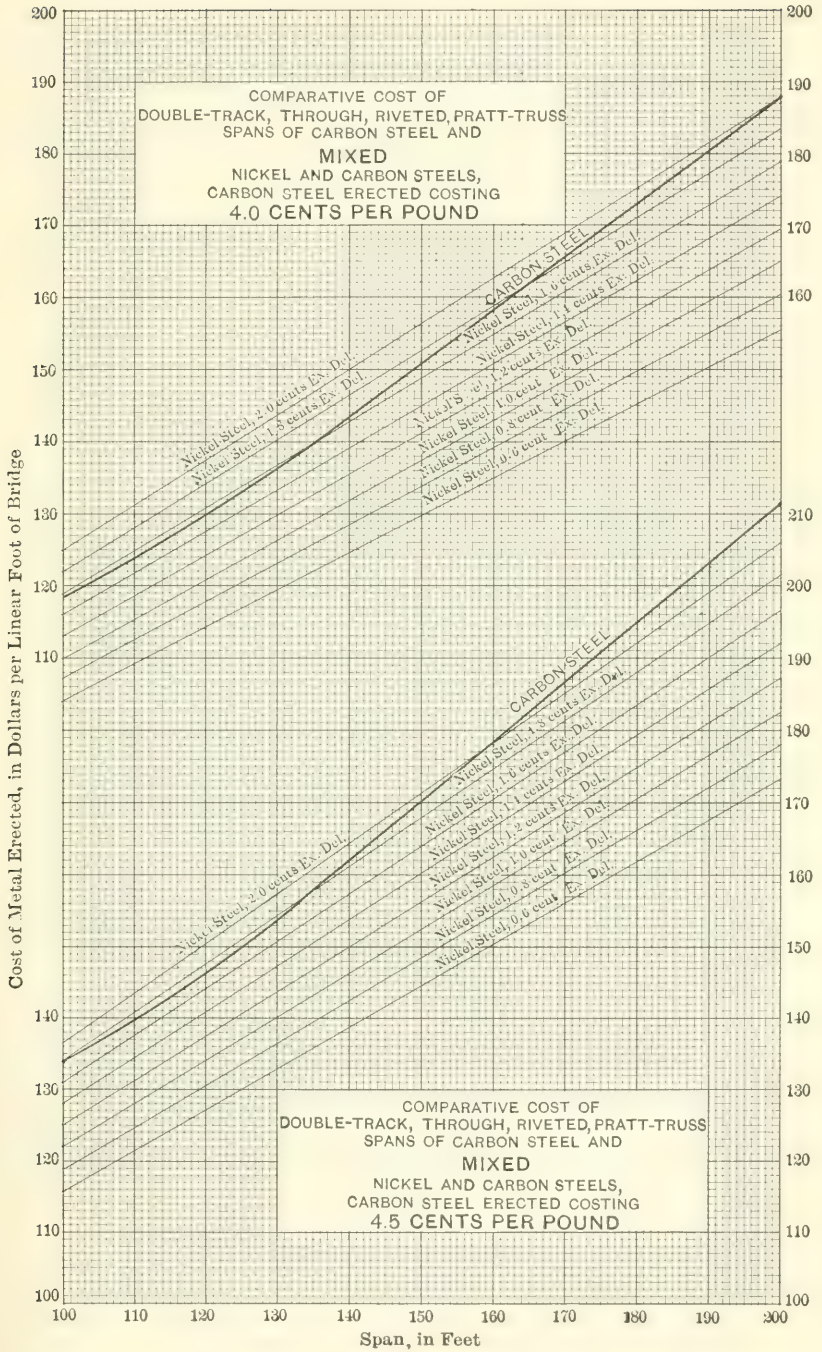

COMPARATIVE COST OF
SINGLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 48.



FIGS. 49 AND 50.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 1.0 cents Ex. Del.
Nickel Steel, 1.1 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 1.3 cents Ex. Del.
Nickel Steel, 1.0 cents Ex. Del.
Nickel Steel, 1.1 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Nickel Steel, 2.0 cents Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
Span, in Feet
100 110 120 130 140 150 160 170 180 190 200
100 110 120 130 140 150 160 170 180 190 200 210

FIGS. 51 AND 52.

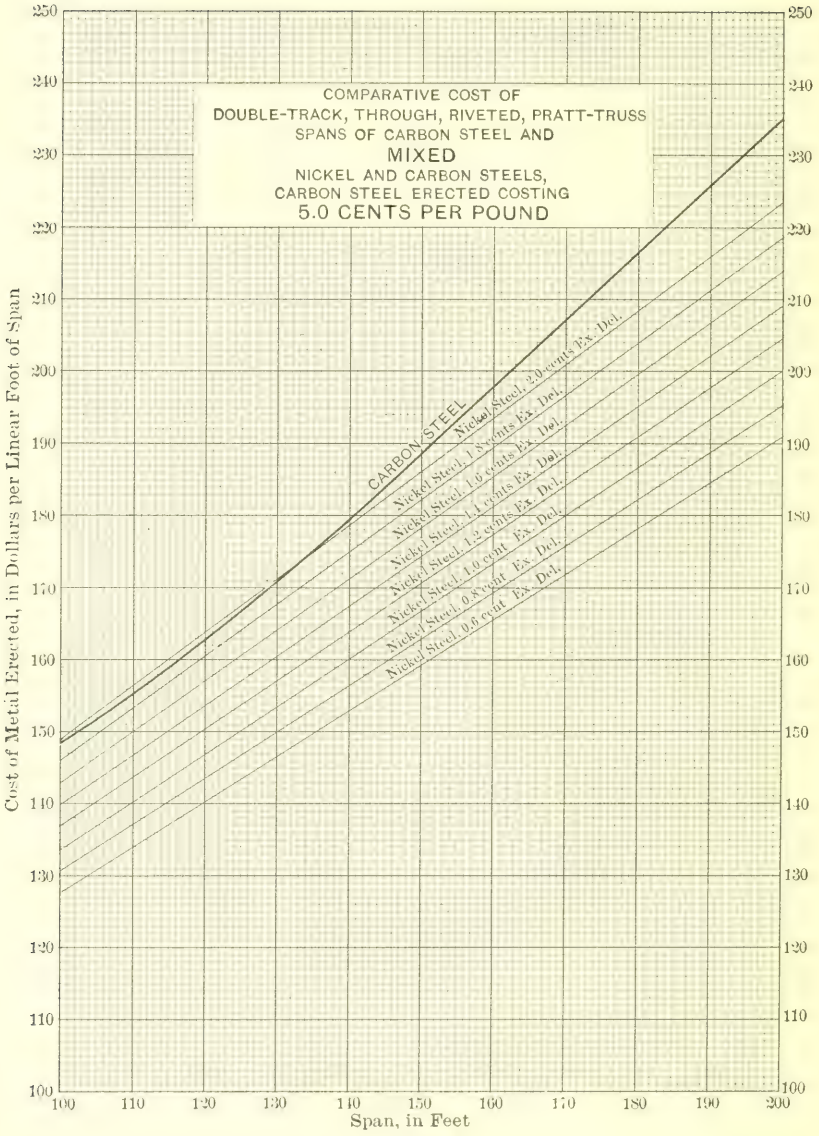

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, RIVETED, PRATT-TRUSS
SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250

FIG. 53.

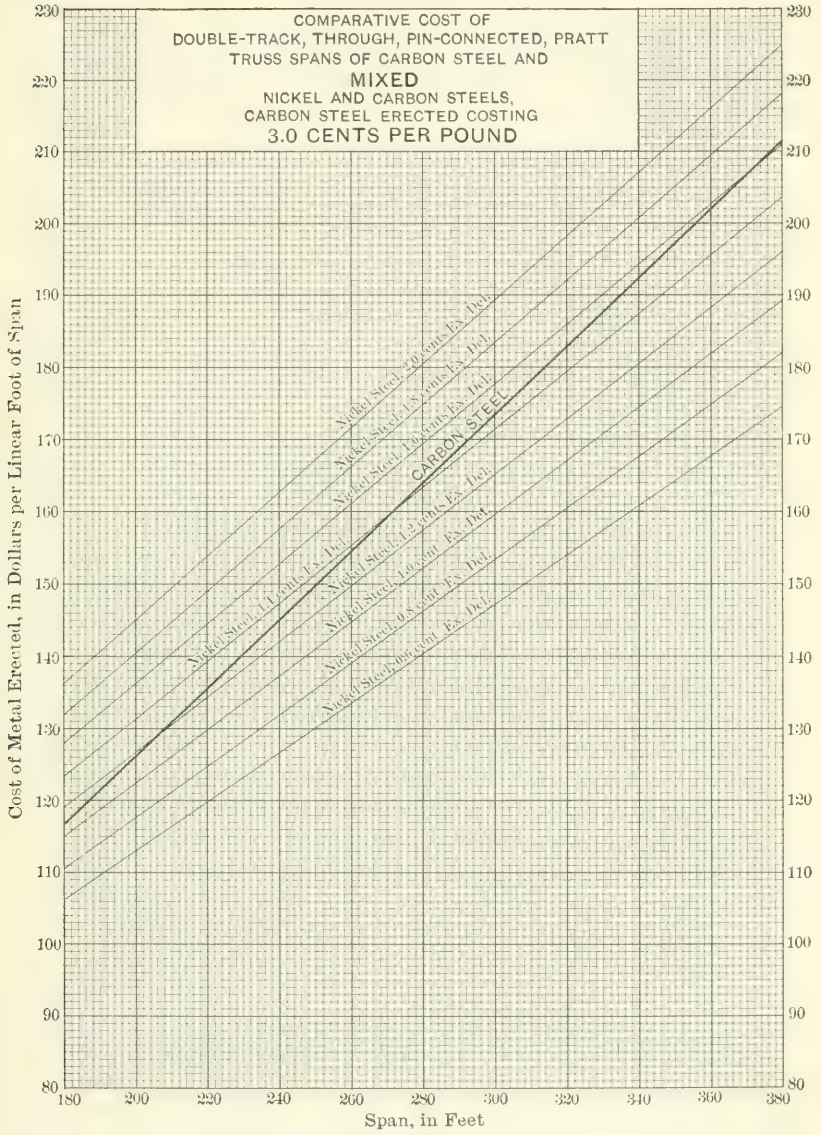

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, PRATT
TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.0 CENTS PER POUND
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
CARBON STEEL
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
230
220
210
200
190
180
170
160
150
140
130
120
110
100
90
80
180
200
220
240
260
280
300
320
340
360
380

FIG. 54.

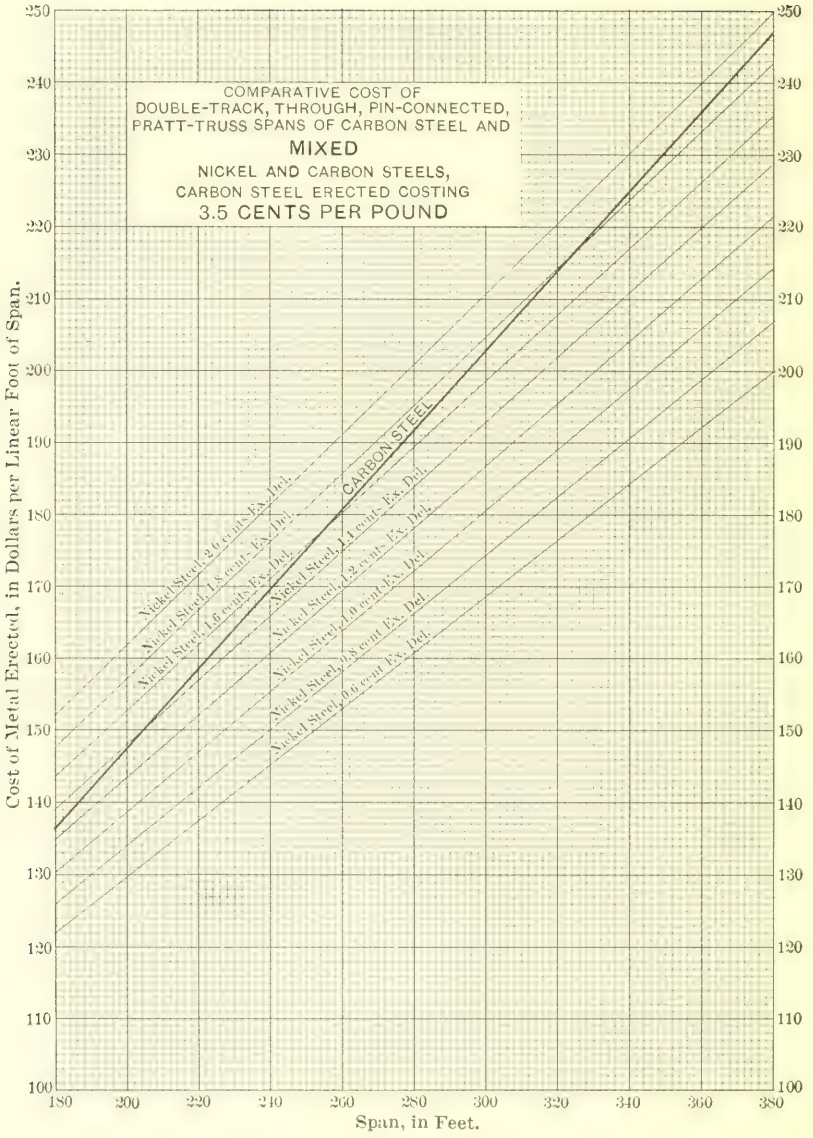

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span.
Span, in Feet.
CARBON STEEL
Nickel Steel, 2.6 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cent Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, .8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.

FIG. 55

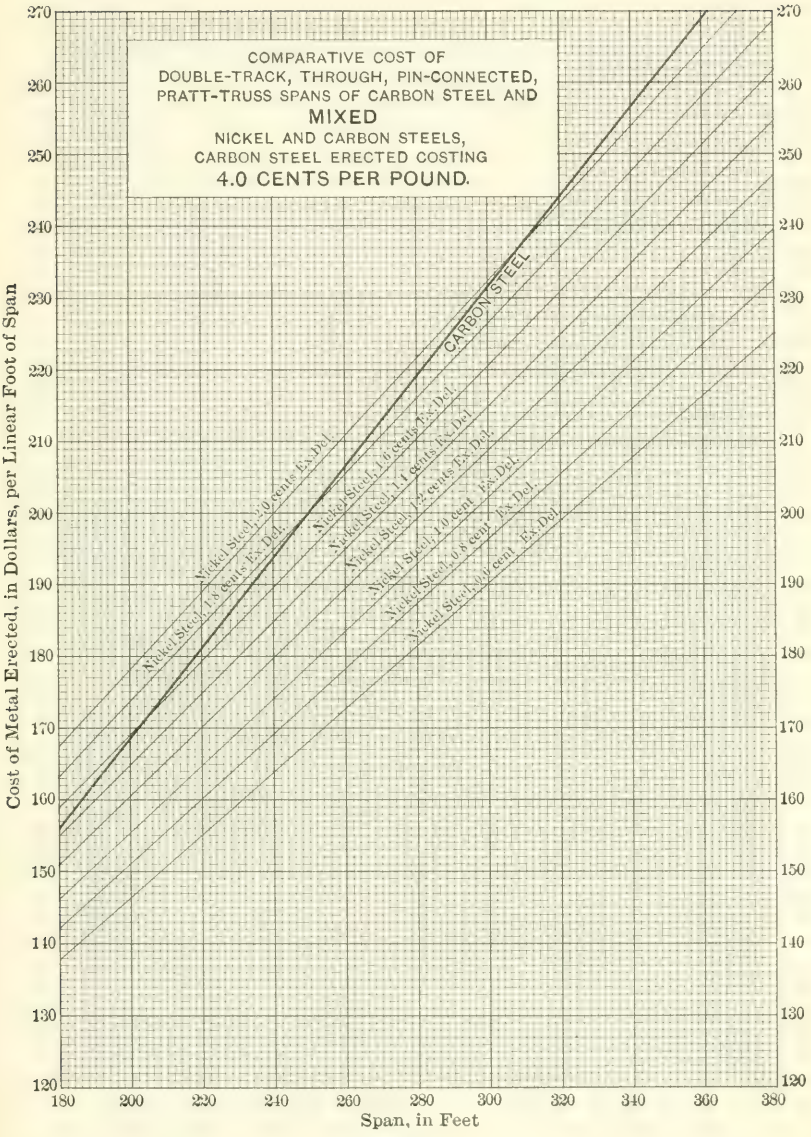

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars, per Linear Foot of Span
Span, in Feet
270
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
180
200
220
240
260
280
300
320
340
360
380

FIG. 56.

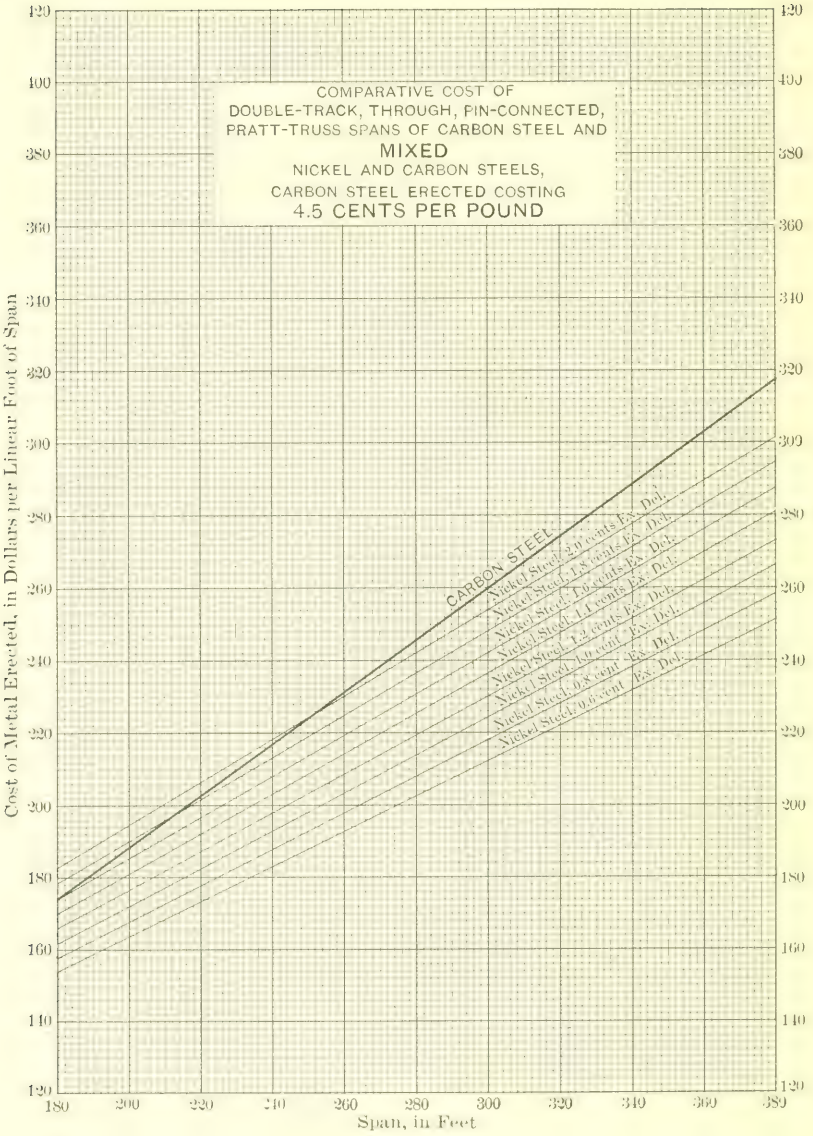
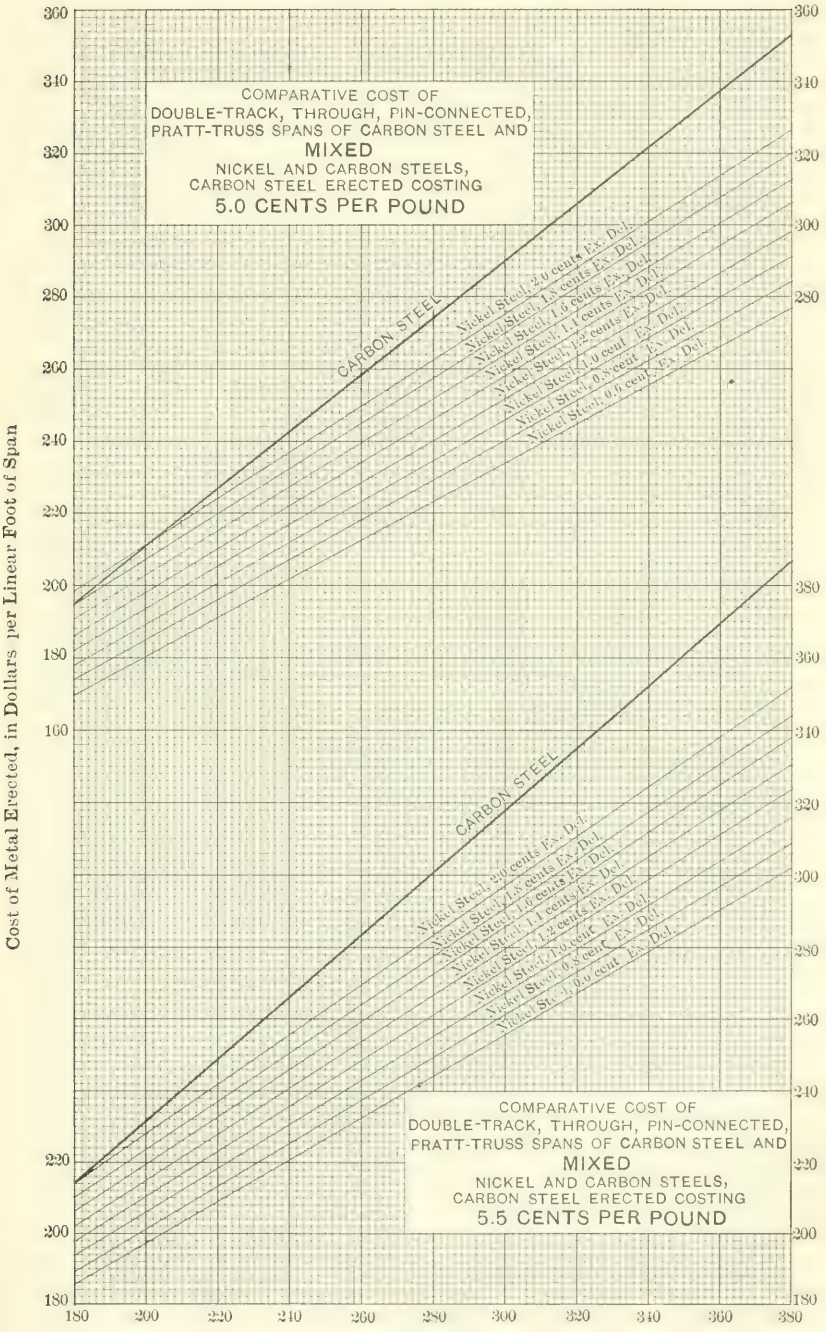
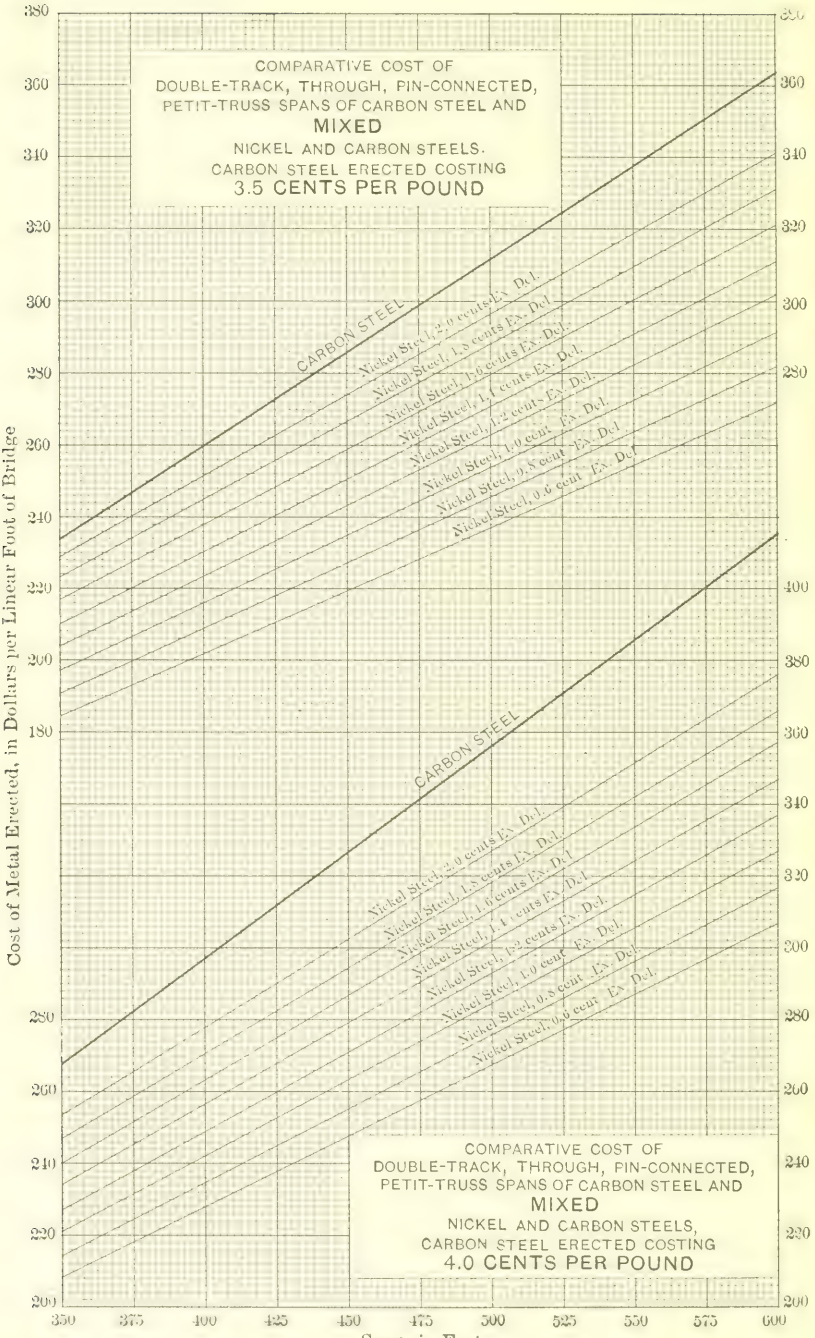

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Span
Span, in Feet
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.

FIG. 57.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PRATT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Lineal Foot of Span
180 200 220 240 260 280 300 320 340 360 380
Span, in Feet

FIGS. 58 AND 59.



FIGS. 60 AND 61.

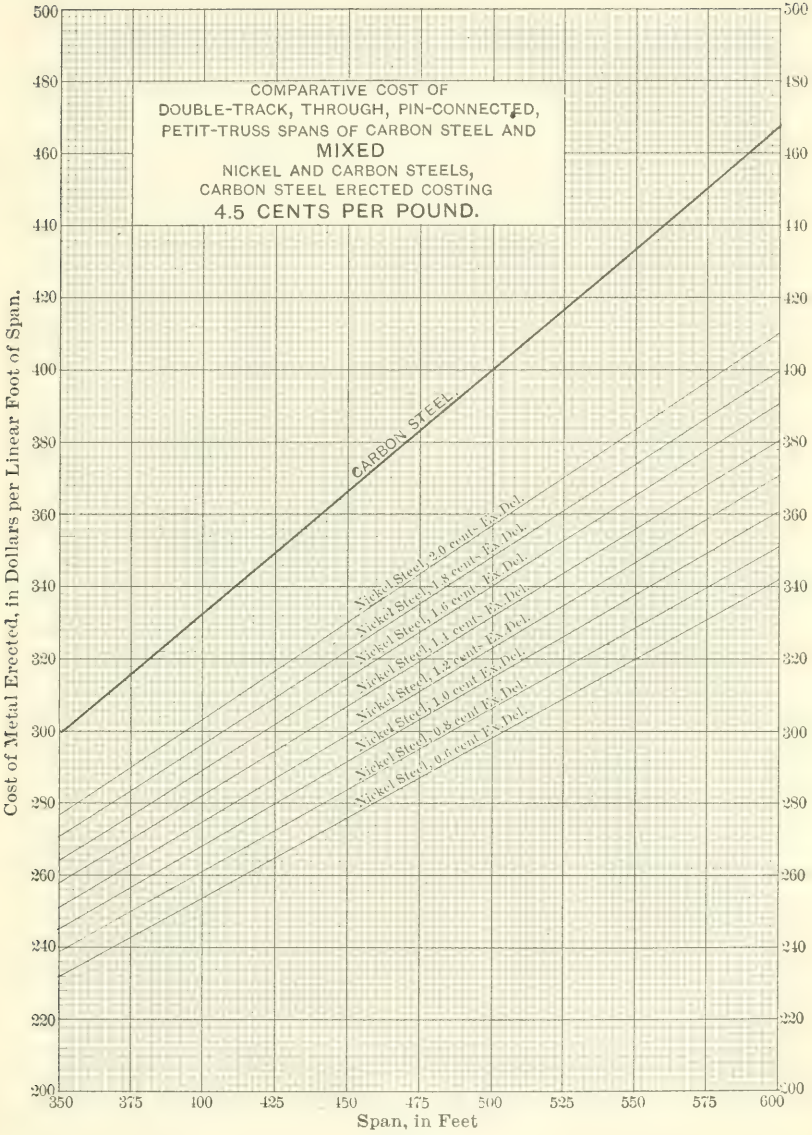

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND.
CARBON STEEL.
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span.
500
480
460
440
420
400
380
360
340
320
300
280
260
240
220
200
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 62.

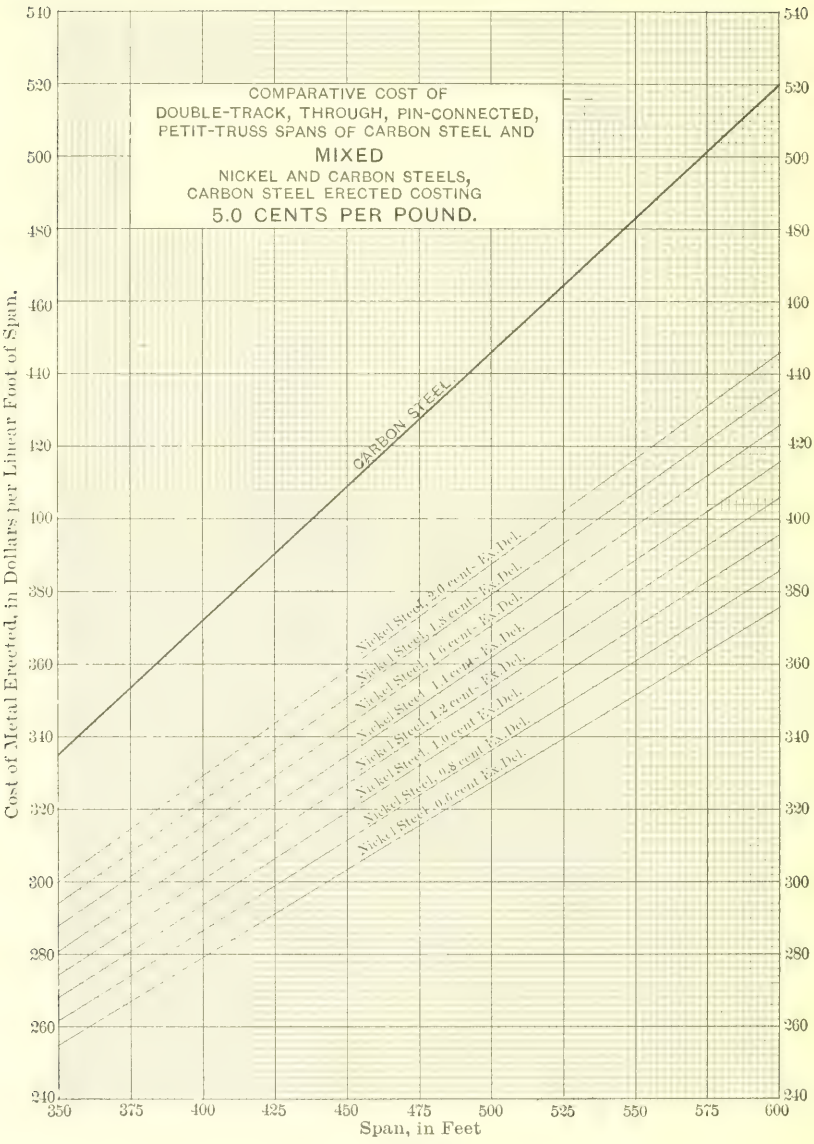

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND.
CARBON STEEL.
Nickel Steel, 2.0 cent. Ex. Del.
Nickel Steel, 1.8 cent. Ex. Del.
Nickel Steel, 1.6 cent. Ex. Del.
Nickel Steel, 1.4 cent. Ex. Del.
Nickel Steel, 1.2 cent. Ex. Del.
Nickel Steel, 1.0 cent. Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Span.
540
520
500
480
460
440
420
400
380
360
340
320
300
280
260
240
350
375
400
425
450
475
500
525
550
575
600
Span, in Feet

FIG. 63.

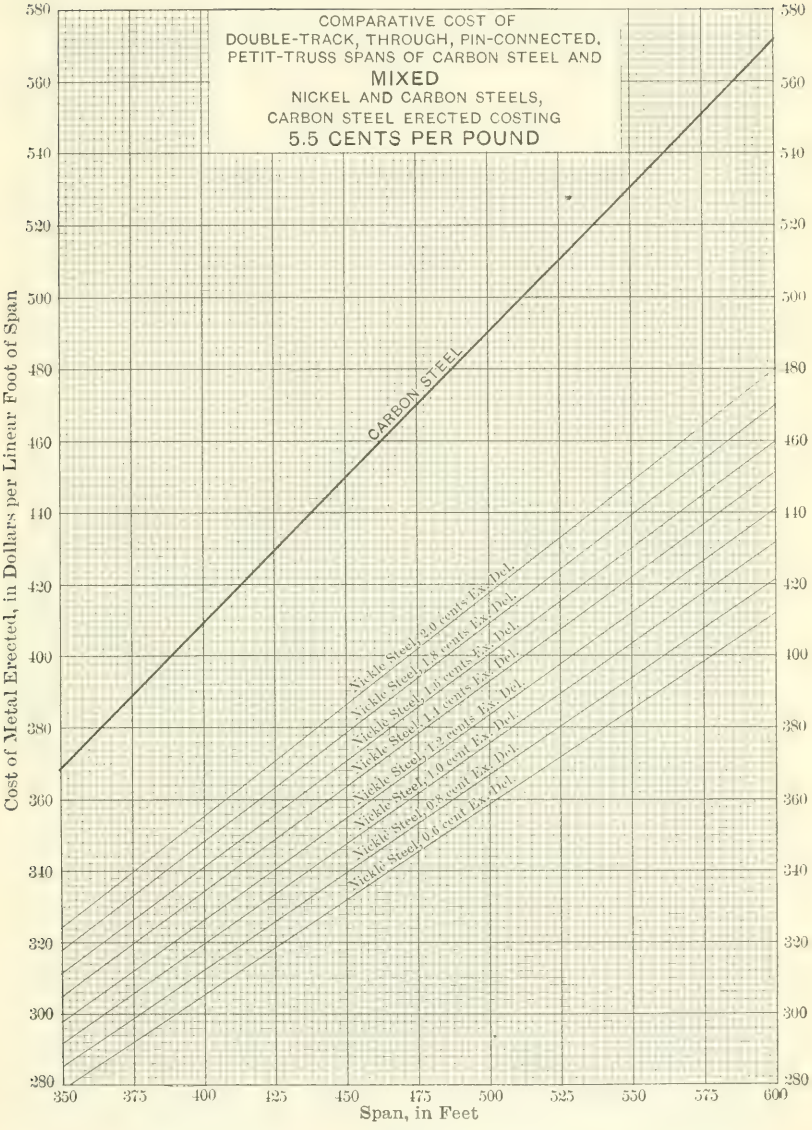
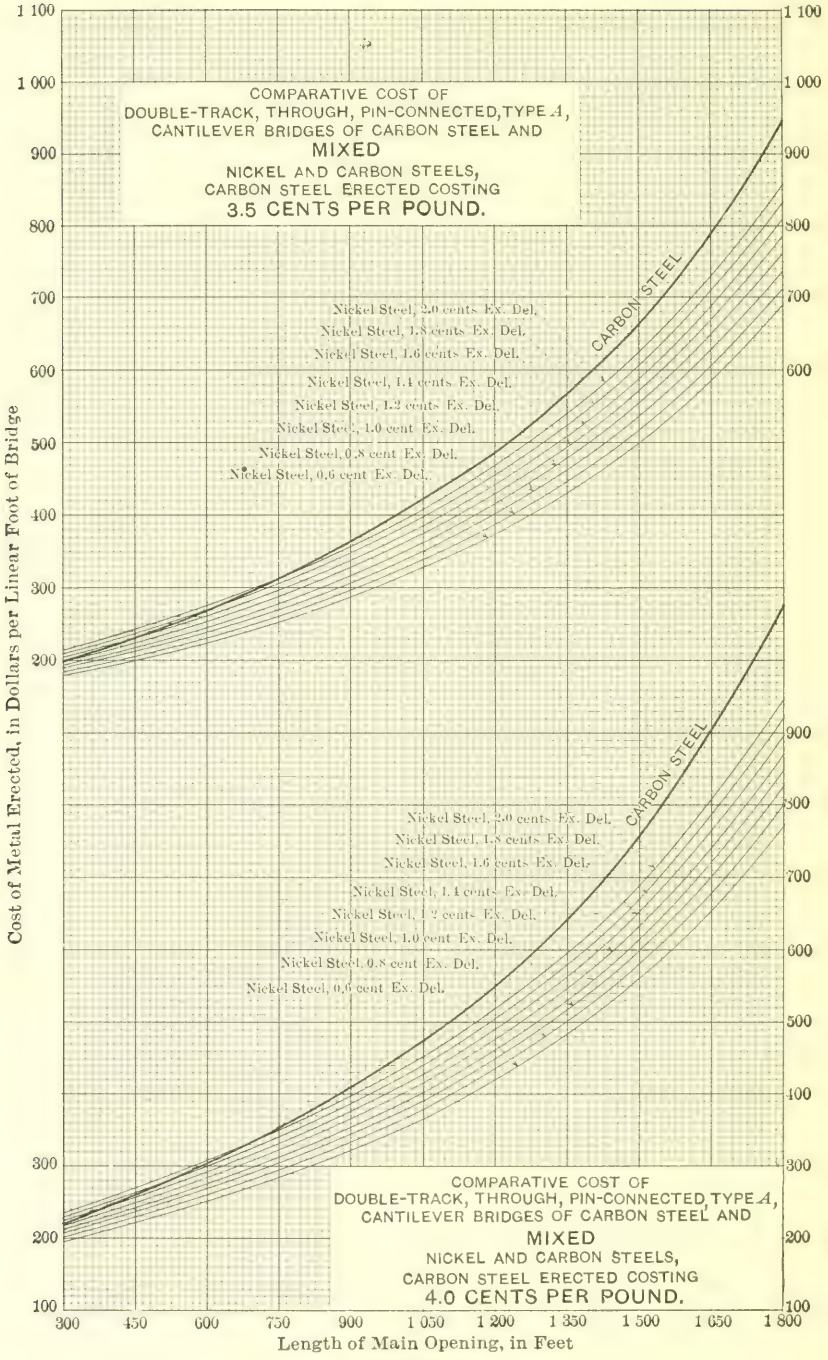
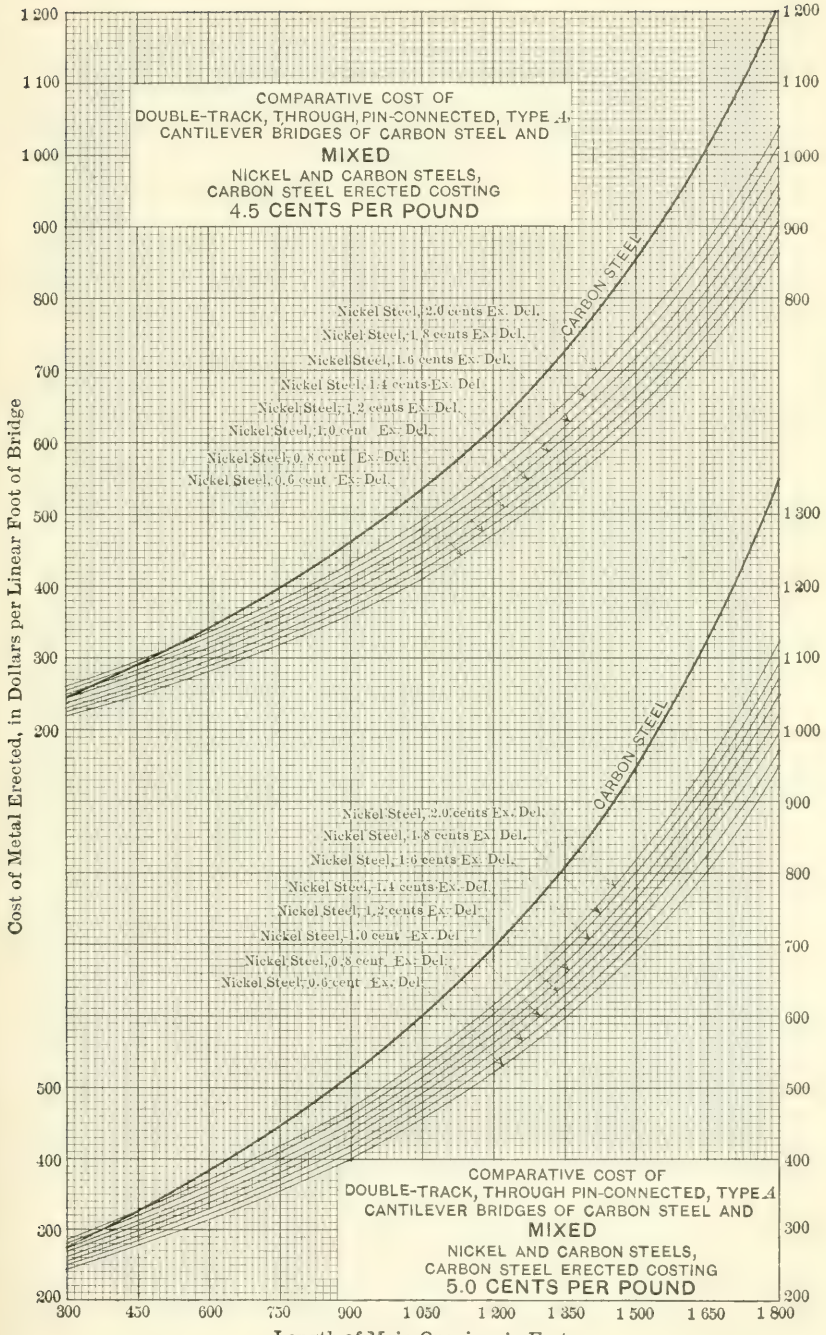

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED,
PETIT-TRUSS SPANS OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
CARBON STEEL
Nickle Steel, 2.0 cents Ex. Del.
Nickle Steel, 1.8 cents Ex. Del.
Nickle Steel, 1.6 cents Ex. Del.
Nickle Steel, 1.4 cents Ex. Del.
Nickle Steel, 1.2 cents Ex. Del.
Nickle Steel, 1.0 cent Ex. Del.
Nickle Steel, 0.8 cent Ex. Del.
Nickle Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Lineal Foot of Span
Span, in Feet
580
560
540
520
500
480
460
440
420
400
380
360
340
320
300
280
350
375
400
425
450
475
500
525
550
575
600

FIG. 64.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A,
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
3.5 CENTS PER POUND.
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A,
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.0 CENTS PER POUND.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
1 100
1 000
900
800
700
600
500
400
300
200
100
300 450 600 750 900 1 050 1 200 1 350 1 500 1 650 1 800
Length of Main Opening, in Feet

FIGS. 65 AND 66.


COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
4.5 CENTS PER POUND
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cents Ex. Del.
Nickel Steel, 1.2 cents Ex. Del.
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
CARBON STEEL
COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH PIN-CONNECTED, TYPE A
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.0 CENTS PER POUND
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
1 200
1 100
1 000
900
800
700
600
500
400
300
200
1 300
300 450 600 750 900 1 050 1 200 1 350 1 500 1 650 1 800
Length of Main Opening, in Feet,

FIGS. 67 AND 68.

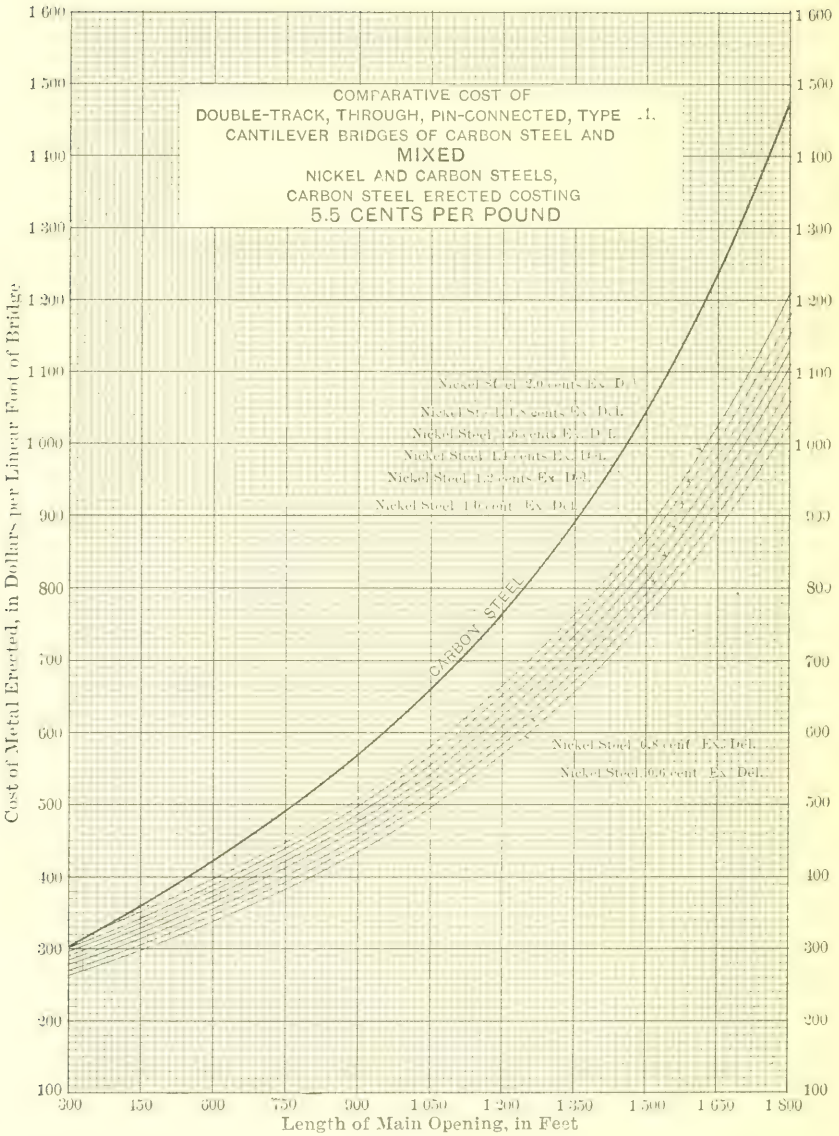

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE M.
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
5.5 CENTS PER POUND
Nickel Steel 2.0 cents Ex. Del.
Nickel Steel 1.8 cents Ex. Del.
Nickel Steel 1.6 cents Ex. Del.
Nickel Steel 1.4 cents Ex. Del.
Nickel Steel 1.2 cents Ex. Del.
Nickel Steel 1.0 cent Ex. Del.
CARBON STEEL
Nickel Steel 0.8 cent Ex. Del.
Nickel Steel 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
500
400
300
200
100
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 69.

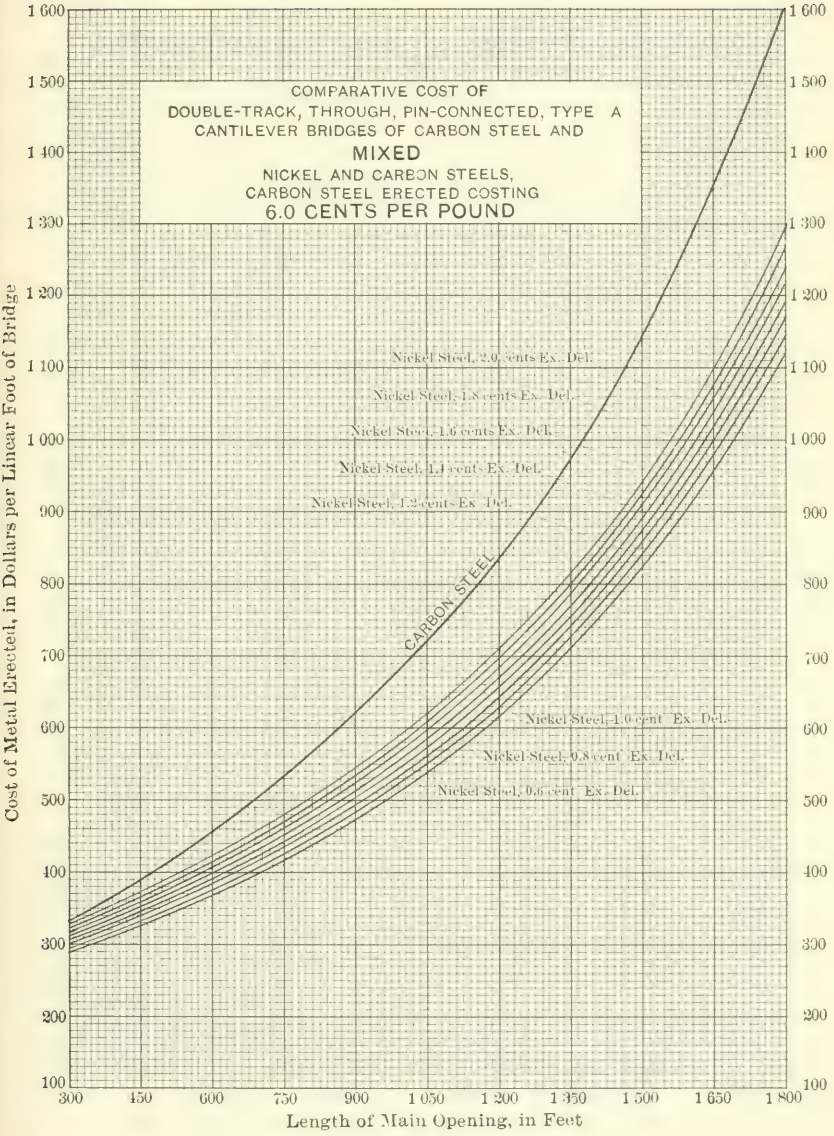

COMPARATIVE COST OF
DOUBLE-TRACK, THROUGH, PIN-CONNECTED, TYPE A
CANTILEVER BRIDGES OF CARBON STEEL AND
MIXED
NICKEL AND CARBON STEELS,
CARBON STEEL ERECTED COSTING
6.0 CENTS PER POUND
Nickel Steel, 2.0 cents Ex. Del.
Nickel Steel, 1.8 cents Ex. Del.
Nickel Steel, 1.6 cents Ex. Del.
Nickel Steel, 1.4 cent Ex. Del.
Nickel Steel, 1.2 cent Ex. Del.
CARBON STEEL
Nickel Steel, 1.0 cent Ex. Del.
Nickel Steel, 0.8 cent Ex. Del.
Nickel Steel, 0.6 cent Ex. Del.
Cost of Metal Erected, in Dollars per Linear Foot of Bridge
1 600
1 500
1 400
1 300
1 200
1 100
1 000
900
800
700
600
500
400
300
200
100
300
450
600
750
900
1 050
1 200
1 350
1 500
1 650
1 800
Length of Main Opening, in Feet

FIG. 70.

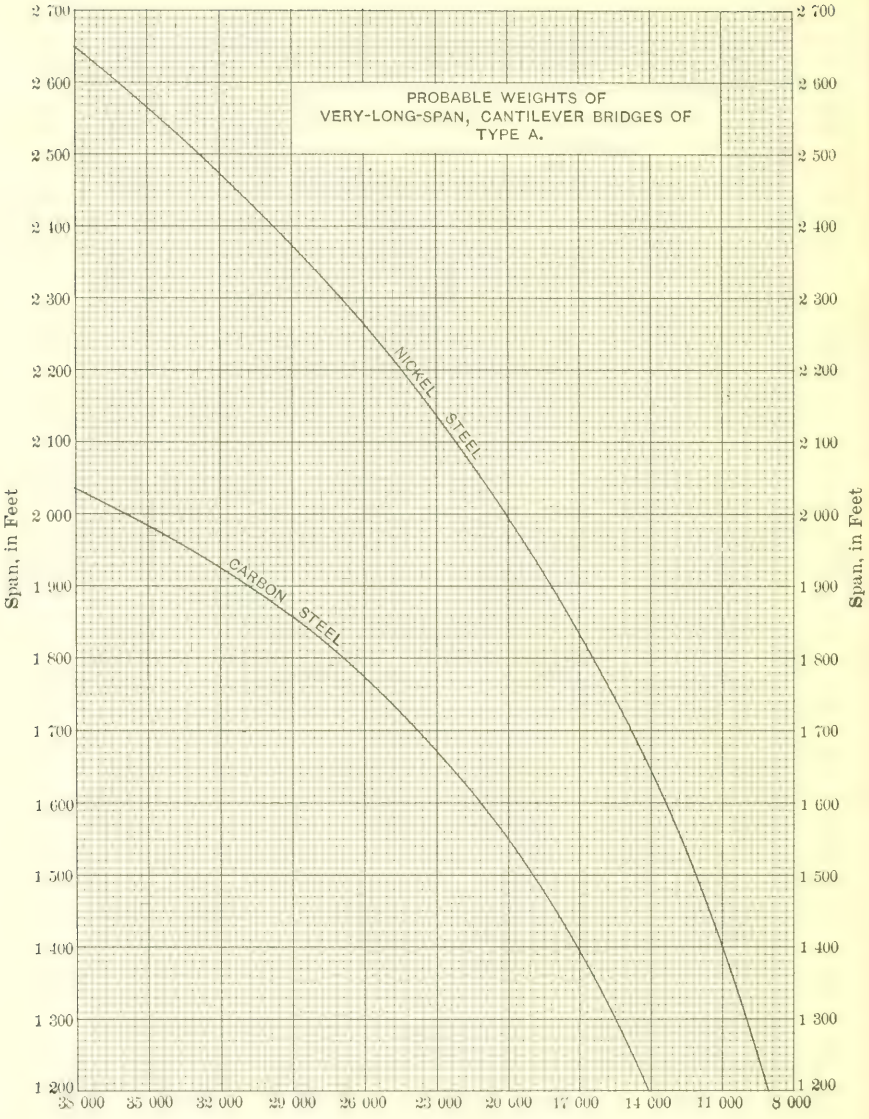

PROBABLE WEIGHTS OF
VERY-LONG-SPAN, CANTILEVER BRIDGES OF
TYPE A.
NICKEL STEEL.
CARBON STEEL.
Span, in Feet
2 700
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
38 000
35 000
32 000
29 000
26 000
23 000
20 000
17 000
14 000
11 000
8 000
Weight of Metal, in Pounds per Linear Foot of Bridge.

FIG. 71.

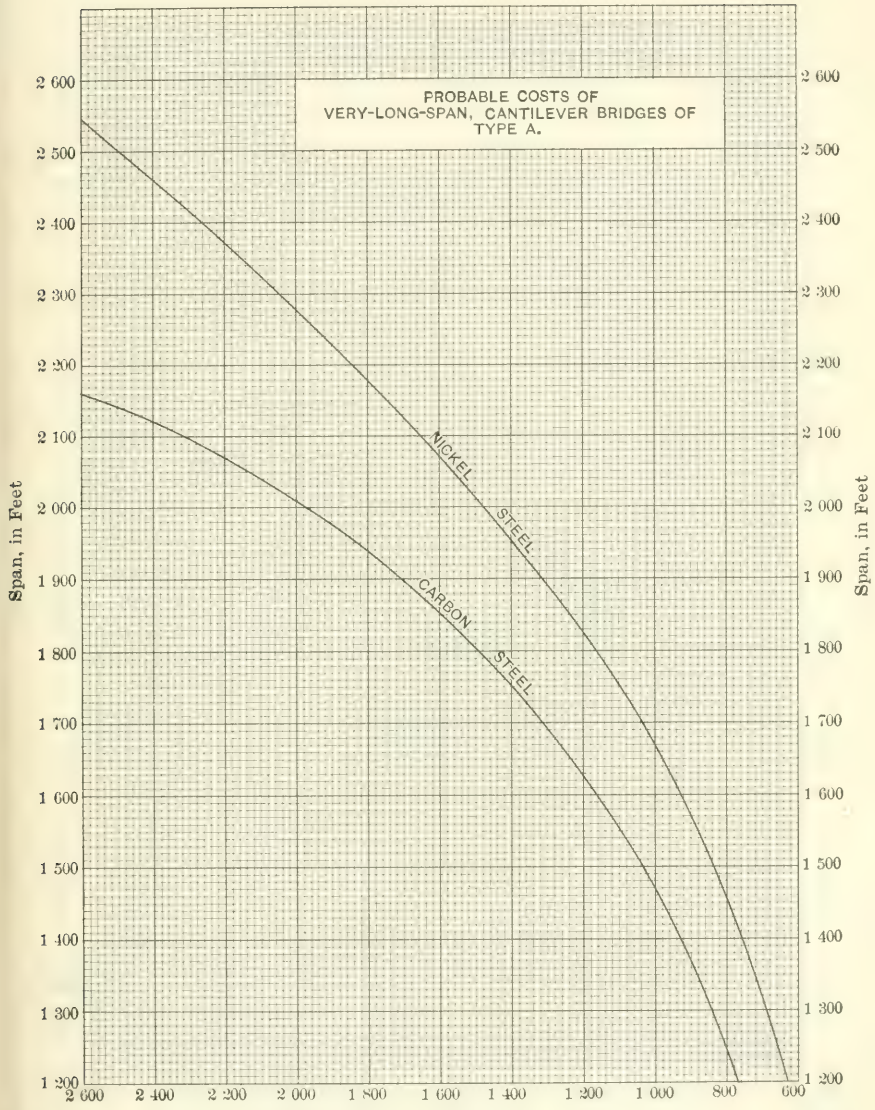

PROBABLE COSTS OF
VERY-LONG-SPAN, CANTILEVER BRIDGES OF
TYPE A.
NICKEL STEEL.
CARBON STEEL.
Span, in Feet
2 600
2 500
2 400
2 300
2 200
2 100
2 000
1 900
1 800
1 700
1 600
1 500
1 400
1 300
1 200
2 600
2 400
2 200
2 000
1 800
1 600
1 400
1 200
1 000
800
600
Cost of Metal Erected, in Dollars per Linear Foot of Span.

FIG. 72.

APPENDIX A. PART I.

COMPARATIVE TESTS OF STRUCTURAL NICKEL STEEL AND MEDIUM CARBON STEEL.

Preliminary Work.—In November, 1903, this investigation had its inception, and, early in December, two parallel series of tests were made at the American Bridge Company's plant at Pencoyd, Pa. For this preliminary work, two steels of the following composition were used:

	Heat No. 16 080.	Heat No. 17 065.
Nickel	3.21	4.25
Carbon	0.390	0.463
Manganese	0.65	0.67
Sulphur	0.015	0.014
Phosphorus	0.011	0.019

A full report of this examination was made in January, 1904, and the results were so promising that immediate steps were taken for a more elaborate series of tests upon a steel made especially for this purpose. Arrangements were made with the Carnegie Steel Company, in the same month, for two melts of nickel steel.

After many delays it was finally decided to have the "shape" nickel steel rolled and tested before ordering the "eye-bar" nickel steel. Two melts of almost identical composition were made at the Homestead Works, Carnegie Steel Company; the first, Heat No. 17 673, on November 1st, and the second, Heat No. 17 749, on December 1st, 1904, each being of basic open-hearth steel.

The following material was shipped to Pencoyd, Pa., for testing during December, 1904, and January, 1905:

1	Universal plate,	12 by $\frac{3}{8}$ in.,	length	17 ft.	
1	"	" 12 by $\frac{1}{2}$ in.,	"	17 ft.	7 in.
1	"	" 12 by $\frac{3}{4}$ in.,	"	15 ft.	1 in.
1	"	" 12 by 1 in.,	"	17 ft.	
5	angles,	6 by 6 by $\frac{3}{4}$ in.,	"	15 ft.	
1	"	8 by 8 by 1 in.,	"	15 ft.	

At the same time other material was shipped to Ambridge, Pa., for the fabrication of struts for compression tests and of eye-bars for "full-sized" tests.

The medium-carbon steel plates for comparative tests were not received at Pencoyd until December, 1905. They were as follows:

1	Universal plate,	12 by $\frac{3}{8}$ in.,	length,	20 ft.	$\frac{1}{2}$	in.
1	"	" 12 by $\frac{1}{2}$ in.,	"	20 ft.	1	in.
1	"	" 12 by $\frac{3}{4}$ in.,	"	19 ft.	$11\frac{1}{2}$	in.
1	"	" 12 by 1 in.,	"	19 ft.	$8\frac{1}{2}$	in.

There were no tests of angles of carbon steel.

Composition of Steels.—Several analyses were made of both the nickel-steel and carbon-steel heats, as shown in Table 7.

TABLE 7.—ANALYSES OF NICKEL AND CARBON STEEL.

NICKEL STEEL. HEAT NO. 17 673.

	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.	Analyst.
Desired.	3.50	0.38	0.70	below 0.04	below 0.03	below 0.04	
1	3.66	0.37	Grav. 0.80	0.025	0.010	0.050	Mill.
2	3.64	0.37	" 0.80	0.023	0.010	0.047	"
3	3.66	0.36	" 0.76	0.024	0.010	0.050	"
4	3.68	0.408 / 0.407	Comb. 0.758	0.020	0.005 / 0.005	Grav. 0.046 Vol.	Shimer, Bethlehem.
5	3.87	0.471	0.771	0.019	0.010	0.039	Booth, Garrett and Blair.
6	3.92	0.417	.....	0.021	0.006	0.035	Orford Works.

Nos. 1, 2, and 3 were ladle analyses, made at the mill during pouring, in the order named; No. 1 between the first and second ingots, No. 2 between the fourth and fifth ingots, No. 3 between the ninth and tenth ingots. The carbon was probably determined by gravimetric analyses.

No. 4 was made from drillings taken from test ingot No 2.

Nos. 5 and 6 were made from duplicate drillings taken from ¾-in. plate.

NICKEL STEEL. HEAT NO. 17 749.

1	3.50	0.36 color.	0.82	0.027	0.011	0.060	Mill.
2	3.50	0.39 comb.	0.78	0.033	0.015	0.060	"
3	3.50	0.38 color.	0.81	0.028	0.011	0.048	"
4	3.52	0.356 comb.	0.763	0.030	0.012	0.058	Shimer.

Nos. 1, 2, and 3 were ladle analyses, made at the mill during pouring, in the order named; No. 1 between the second and third ingots, No. 2 between the tenth and eleventh ingots, No. 3 between the twelfth and thirteenth ingots.

No. 4 was made from drillings taken from test ingot No. 2.

CARBON STEEL. HEAT NO. 33 342.

1	.....	0.253	0.546	0.025	0.014	0.016	Booth, Garrett and Blair.
2	.....	0.198	0.55	0.025	0.011	0.012	Orford Works.

Nos. 1 and 2 were made from duplicate drillings taken from ¾-in. plate.

CARBON STEEL. HEAT NO. 41 520.

1	.....	0.19	0.54	0.033	0.022	....	Mill.
2	.....	0.287	0.563	0.025	0.025	0.020	Booth, Garrett and Blair.
3	.. ..	0.234	0.62	0.024	0.023	0.035	Orford Works.

Nos. 2 and 3 were made from duplicate drillings taken from ¾-in. plate.

For use in the fabrication of struts and for the 12-in. universal plates, tested for comparison with the nickel steel, a medium carbon steel of good quality was required.

All the 12-in. plates for specimen tests were rolled from Nickel-Steel Heat No. 17 673 and Carbon-Steel Heat No. 33 342. The angles for specimen tests and the greater part of the material for the struts were rolled from Nickel-Steel Heat No. 17 749 and Carbon-Steel Heat No. 41 520.

A comparison of the analyses of the steels shows that the material obtained was as close to the desired composition as it was practicable for the mill to make. The tests made on the plates did not reveal any lack of homogeneity.

The surface of the nickel-steel plates was smooth and free from scale, much more so than that of the carbon-steel plates. The surface of the nickel-steel angles was not so good, as in one or two instances the fin from the edge and some scale had been rolled in.

All the material shipped to Pencoyd and Ambridge was stored unprotected from the weather for several months, the best means for identification later, being the cleaner and smoother surface of the nickel steel.

Tests of 12-in. Universal Rolled Plates and Angles.

Tensile Tests—Plain Specimens.—The test given the greatest attention in mill practice everywhere is the tensile test. In the absence of inspection, or when, for any reason, knowledge of the physical properties of material is desired by the mill, a tensile test only is made. At many mills no other test is made by the inspector for his client. For these reasons, a large number of specimens were prepared from this structural nickel steel and from the structural carbon steel tested for comparative purposes.

Because of the uncertainty attached to the usual method of determining the "yield point" or so-called "elastic limit," arrangements were made for two series of tests, one on parallel-sided pieces and one on pieces having edges machined parallel for a distance of $9\frac{1}{2}$ in., only, with fillet widening pieces at the ends, and known as the standard adopted by the American Association of Steel Manufacturers (abbreviated here into A. A. S. M.). The length of all specimens was 18 in., and the width, except for those cut from the 1-in. material, was $1\frac{1}{2}$ in. The width of these was reduced to $1\frac{1}{4}$ in. because it was expected that a 150 000-lb. machine would be used to make the tests, and it was necessary to keep the ultimate strength well within this limit.

The "parallel-sided" pieces were to be tested according to mill practice at a representative mill. At the suggestion of Mr. C. L. Huston, of the Lukens Iron and Steel Company, Coatesville, Pa., half these pieces were tested at this mill, and the others at the Pencoyd Iron Works,

Pencoyd, Pa. Mill practice was not followed exactly, because it was not thought possible to get even a close approximation for the "yield point" of the nickel steel, with the high speeds in every-day use. The only change, however, was in the reduction of the speed of breaking.

The A. A. S. M. pieces were to be tested in a machine equipped with an autographic attachment; and a 200 000-lb. Olsen machine, at Drexel Institute, Philadelphia, was used, through the kindness of Mr. Earl B. Smith, Instructor in Mechanical Engineering. All the machine work was done at the Pencoyd Iron Works.

The tensile tests in the original lay-out were located at the ends of the plates and angles, the A. A. S. M. pieces at the edge, and the parallel-sided pieces inside and adjacent. As it was desirable to repeat certain tests to bring out more definitely points of difference between the two steels, additional pieces had to be selected from available material. The tensile strength of plates varies with the location of the specimen in the plate and of the slab in the ingot, so this forced selection of additional pieces added necessary information on these points. All the material was rolled from slabs; hence these variations in physical properties due to location are not so pronounced as in material rolled directly from the ingot.

American Association of Steel Manufacturers' Specimens.—The results of these tests are given in detail in Tables 30 and 31, and a comparison with the results of tests on parallel-sided specimens may be found in that part of this appendix filed for reference in the Library of the Society.

The speed of machine or movement of head for the first eleven tests was 1 in. in 6 min., but, while this gave good results for the carbon steel, it was too rapid for the nickel steel, and the yield point was lost. The weight could not be made to travel along the beam with sufficient rapidity to keep the beam balanced, and, consequently, the pencil recorded a faulty curve. A speed of 1 in. in 6 min. was desired in order to economize time, the speed, 1 in. in 20 min., adopted for the remaining pieces, requiring $\frac{1}{2}$ hour for each test. In these remaining tests, up to a point just below the yield point, a speed of 1 in. in 6 min. was used, and then a speed of 1 in. in 20 min. until the material had well passed this point, when 1 in. in 6 min. was again used. In several tests, when the maximum strength had been developed, a still faster speed (1 in. in 2 min.) was used for breaking the specimen. The speed at breaking was found to affect the tensile strength materially.

Yield Point.—The term "yield point" has been adopted herein in place of "elastic limit" because it more nearly represents what was sought, namely, the point at which the material would no longer return to its original condition upon the removal of the load. It is shown graphically in the autographic diagram by the departure of the record from a straight line.

The determination of the true elastic limit was not possible with the apparatus at hand.

The loss of time caused by the use of an autographic attachment bars the adoption of this method in mill practice, yet it was desired that the yield point reported in these tests should be a point easily obtainable in practice by a skillful operator, and not one depending upon time-consuming and expensive devices for its exact location. It was assumed to be the "least permanent set" visible in a specimen by the aid of a pair of finely pointed dividers. The point in the curves corresponding to this "least permanent set" was obtained after some experimenting, as shown in Table 8.

TABLE 8.—Beam Readings.

Mark.	Drop of beam, in pounds.	Yield point, in pounds.	Load released at:	Permanent set in 6 in. by dividers:
CTSL 150	38 600	35 200	36 600 lb.	0.01 in.
" 193	36 600	34 800	36 000 "	0.02 "
" 198	39 400	33 800	33 200 "	0.01 "
" 152	34 200	33 000	33 600 "	0.015 "
TSL 3	.....	38 600	38 000 "	0.005 "
" 40	.....	39 200	39 000 "	0.015 "
" 43	.....	40 600	40 000 "	0.01 "
" 44	.....	38 800	39 000 "	0.015 "
" 52	.....	46 500	48 000 "	0.02 "
" 70	.....	47 100	46 900 "	0.035 "
" 89	.....	47 500	48 000 "	0.02 "
" 109	.....	70 500	69 300 "	0.00 "
			75 000 "	Between these readings a rapid elongation of specimen took place.
" 150	.....	77 400	77 800 "	0.025 in.
" 154	.....	76 600	77 400 "	Appreciable.
" 199	.....	73 000	72 000 "	Not appreciable.
			76 000 "	0.01 in.
ATSL 58	.....	69 200	64 400 "	Not appreciable.
	.....	.....	70 600 "	Not appreciable.
	.....	.....	75 000 "	0.02 in.

In the tests given in Table 8 the load was removed at about the yield point, the permanent set, if any, was measured, and the movement of the pencil noted. If this traced a new line parallel to the original curve, there was visible evidence of a permanent set and of the amount. To check this, careful measurements were made by dividers. It was assumed that, for all practical purposes, a set of 0.01 in. in 8 in. was the "least visible," and the point in the curve corresponding to this was obtained graphically at the intersection of a line 0.01 in. away from, and parallel to, the straight portion of the record. The stretch shown by the curve is the stretch in 8 in. It was found that, even when the record had departed from a straight line, if approximately this distance of 0.01 in. had not been exceeded, on releasing the load the pencil would retrace the original line, and no elongation could be detected by the dividers.

At times the mechanism of the recorder did not work properly, but usually these vagaries were easily corrected. The movements of the weight and of the pencil were constantly checked, one with the other. The yield point thus determined is slightly lower than the drop of beam. In tests of nickel steel, of this content of nickel and carbon, the beam does not drop at the point where the stretch continues without an increasing load, because there is no such point. The autographic curves show this; for it will be seen that in no curve of nickel steel is there a bend at the yield point of even approximately 90 degrees. A "drop" will occur if the weight is run out uniformly, for the rate of stretch changes, but it will be almost invariably at too great a load as registered by the weight on the beam, for in mill practice the speed is so great that the beam is never balanced until after the so-called elastic limit or yield point is reached. In carbon steel an opportunity is given, by the stretching of the steel, for the weight to "catch up"; in nickel steel there is none. This subject will be mentioned later.

The elongations in 2, 4 and 6 in. are given; although their value is not great, they show that considerable stretching takes place over the entire length of the specimen.

The "area of fractured section" was measured at points midway between edges at Pencoyd and Drexel. This method is followed in most mills, but it gives slightly higher percentages of reduction than actually exist. At Lukens, edge measurements were taken, that being the practice at that mill. The dimensions of this section, and also of the original section, are given to the nearest 0.005 in., this being near enough for all practical purposes. At Pencoyd and Drexel the elongations were measured along the edge of the specimen; at Lukens, along the middle. This difference affects slightly the comparison of results. The elastic ratio is the ratio of yield point or drop of beam to the ultimate strength.

In Table 31 both drop of beam and yield point, for specimens broken at Drexel Institute, are given. The "stretch at the drop of beam," in Table 31, has no special significance, except to show why mill methods will give fair results for testing carbon steel and not for nickel steel, in which no such stretch occurs. It was measured from the autographic records. The drop of beam was taken from the cards as that load at which the curve first became horizontal. Actually, no drop of beam occurred, as the machine was equipped with a device for controlling automatically the movement of the weight; and, when adjusted properly at the slow speed used, the beam was balancing throughout.

Parallel-Sided Specimens.—The tests on parallel-sided specimens are interesting, as showing how close an approximation to the true yield point may be obtained by the drop of the beam, by expert operators of mill-testing machines.

A 200 000-lb. Olsen machine was used at each mill. At Pencoyd the weight was run out on the beam by an electric motor, excited by contacts at the end of the beam, and, at Lukens, by a hand-wheel. Operating by hand is shown to be much more sensitive, the reason being easily seen by one witnessing a test. When testing the nickel steel, the Pencoyd operator had to know in advance about where to expect the drop. Even with this information, the results are largely guesswork. The Lukens operator did not need this information, and, while the beam readings are a little high, for the reasons explained, the results are in fair agreement with those obtained with the autographic recorder, and the time for making the tests is brought within the possibilities of mill practice. The speeds used at each mill for nickel steel were reduced below those for carbon steels.

EXAMINATION OF RESULTS—NICKEL STEEL—TABLES 30 AND 32.

A fair degree of uniformity exists throughout.

Yield Point.—Edge specimens have a higher yield point than corresponding interior pieces. Specimen tests made at Lukens agree closely with those made at Drexel, but those made at Pencoyd are generally higher. The yield point becomes lower as the thickness of the material increases.

Material..... Speed, 1 in. in:.....	⅜-in. plate. 20 min.	½-in. plate. 20 min.	¾-in. plate. 20 min.	1-in. plate. 20 min.	¾-in. angle. 20 min.	1-in. angle. 20 min.
Edge.....	70 100	.....	62 100	58 600	62 300	54 200
Interior.....	67 100	62 100	.....	58 500	.....	.....

Tensile Strength.—Edge specimens have a tensile strength about 3 000 lb. higher than corresponding interior pieces. All specimens cut from the same thickness agree closely when variations of speed and location are considered. The tensile strength becomes less as the thickness increases, also as the speed of breaking decreases.

TABLE 9.

MATERIAL...	⅜-IN. PLATE			½-IN. PLATE			¾-IN. PLATE.		
Speed, 1 in. in:	6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.
Edge.....	116 600	116 300	.....	110 600	113 800	.....	106 800	108 400	.....
Interior.....	113 900	112 600	116 400	.....	107 500	112 500	.....	108 100	111 200

MATERIAL....	1-IN. PLATE.			¾-IN. ANGLE.			1-IN. ANGLE.		
Edge.....	.....	107 300	.....	103 000	.....	.....	.....	97 900	.....
Interior.....	104 600	103 500	105 700	.....	.....	104 400	.....	.....	100 300

1-in. and $\frac{3}{4}$-in. plates were rolled from slabs $14\frac{1}{2}$ by $9\frac{1}{4}$ in.
$\frac{1}{2}$-in. " $\frac{3}{8}$-in. " " " " " $14\frac{1}{2}$ by $8\frac{1}{4}$ in.
8 by 8 by 1-in. angles " " " " $11\frac{1}{2}$ by 8 in.
6 by 6 by $\frac{3}{4}$-in. " " " " " 8 by 9 in.

The slabs were rolled from ingots 25 by 30 in.

The relative amounts of work done on the slab in rolling are shown in Table 10.

TABLE 10.—Relative Amounts of Work Done in Rolling.

12-in. plates, thickness.	Area of slab.	Area of shape.	Percentage of reduction.
1 in.	134 sq. in.	12 sq. in.	91.0
$\frac{3}{4}$ "	134 "	9 "	93.3
$\frac{1}{2}$ "	120 "	6 "	95.0
$\frac{3}{8}$ "	120 "	$4\frac{1}{2}$ "	96.3
Angles.			
$8 \times 8 \times 1$ in.	92 "	$15\frac{1}{2}$ "	83.1
$6 \times 6 \times \frac{3}{4}$ in.	72 "	$8\frac{1}{2}$ "	88.2

The differences in tensile strength due to thickness alone result probably from a combination of two varying conditions: first, the chilling during rolling; and, second, the amount of reduction of the slab. Differences in other physical properties necessarily arise from these same conditions.

Elastic Ratio.—The elastic ratio is seen to decrease uniformly as the thickness increases, varying from 60.1 for the $\frac{3}{8}$-in. material to 54.5 for the 1-in. material.

Elongation and Reduction of Area.—The elongation and reduction of area increase slightly as the thickness of the material increases, but the variation lacks uniformity.

TABLE 11.—Elongation and Reduction of Area.

Material.....		$\frac{3}{8}$-in. Plate.			$\frac{1}{2}$-in. Plate.			$\frac{3}{4}$-in. Plate.		
Speed, 1 in. in:.....		6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.	6 min.	3-2 min.	40-10 sec.
Elongation:	Edge....	.....	15.7	.....	16.5	16.4	.....	17.5	18.1	.....
	Interior.	15.1	16.5	15.6	.....	17.1	16.5	.....	16.5	17.8
Reduction of Area:	Edge....	.....	46.4	.....	46.9	46.0	.....	48.1	47.4	.....
	Interior.	45.6	48.9	45.9	.....	50.0	47.3	.....	43.6	40.7

Material.....		1-in. Plate.			$\frac{3}{4}$-in. Angle.			1-in. Angle.		
Elongation:	Edge....	.....	20.0	.....	18.7	.....	.....	.....	21.3	.....
	Interior.	19.8	20.5	21.2	.....	.....	17.9	.....	.....	18.4
Reduction of Area:	Edge....	.....	47.4	.....	42.0	.....	.....	.....	48.5	.....
	Interior.	49.2	49.8	48.5	.....	.....	35.2	.....	.....	41.0

Speed of Machine.—The tensile strength is raised by an increase in speed; the effect on elongation and reduction of area of fracture is not marked; the tendency, however, is to reduce the reduction of area.

EXAMINATION OF RESULTS—CARBON STEEL—TABLES 31 AND 33.

From only one thickness of material, 1-in. plate, were specimens cut from both edge and interior, hence remarks as to the effect of location are limited to this one example.

Yield Point.—Edge specimens have a yield point slightly higher than the corresponding interior pieces.

Drop of Beam.—

The average drop of beam for all Drexel tests is 34 100 lb.
" " " " " " " Lukens " " 32 800 "
" " " " " " " Pencoyd " " 40 600 "

The speed of machine differed in each case, being 1 in. in 6 min. at Drexel, 1 in. in 3 min. at Lukens, and 1 in. in 15 sec. at Pencoyd, yet the close agreement between the two former and the great difference of the Pencoyd readings show how little dependence can be put upon results obtained with the machine running at a high speed.

The average of all the Drexel tests for the yield point is 32 800, which is 1 300 lb. lower than the corresponding drop of beam.

There is a close uniformity between the results obtained at the same place.

	$\frac{3}{8}$ in. plate.	$\frac{1}{2}$ in. plate.	$\frac{3}{4}$-in. plate.	1-in. plate.		
Yield point at Drexel	38 600	36 900	33 400	27 800	(26 100)*	Edge.
Drop of beam at Drexel	40 800	37 300	34 100	30 300	(27 000)*	Edge.
Drop of beam at Lukens	38 000	33 900	30 900	28 500	.....	Interior.
Drop of beam at Pencoyd	46 700	40 800	36 400	38 500	.....	Interior.

* Interior specimens.

These figures show how regular is the decrease in the yield point as the thickness of the material increases. The same conditions probably exist as influenced the physical properties of the nickel steel. The edge specimens cut from the 1-in. plate have a higher yield point than the corresponding interior ones.

Tensile Strength.—Edge specimens have a tensile strength about 3 000 lb. higher than the corresponding interior pieces.

MATERIAL...	$\frac{3}{8}$-IN. PLATE.			$\frac{1}{2}$-IN. PLATE.			$\frac{3}{4}$ IN. PLATE.			1-IN. PLATE.		
Speed, 1 in. in:	6 min.	3-2 min.	15-10 sec.	6 min.	3 min.	15-10 sec.	6 min.	3 min.	15-10 sec.	6 min.	3-2 min.	15-10 sec.
Edge		63 700		63 800			62 200			61 900		
Interior		62 400	66 100		62 000	65 300		61 100	62 700	59 000	58 300	61 600

The differences due to thickness are evident, but they are small, and those due to speed of breaking and of location in plate are well marked.

Elastic Ratio.—The elastic ratio decreases as the thickness of material increases, varying from 60.8 for the ⅜-in. material to 45.0 for the 1-in. material.

Elongation and Reduction of Area.—The elongation seems to increase slightly as the thickness increases, but the effect is small, and it is not noticeable at all on the reduction of area. An increase in the speed of breaking causes a decrease in the reduction of area, although this is not well marked, and the effect on the elongation is not noticeable.

Material		⅜-in. Plate.			½-in. Plate.			¾-in. Plate.			1-in. Plate.		
Speed, 1 in. in:		6 min.	3.2 min.	15-10 sec.	6 min.	3 min.	15-10 sec.	6 min.	3 min.	15-10 sec.	6 min.	3.2 min.	15-10 sec.
Elongation	Edge	....	28.0	....	27.4	....	....	22.2	....	....	....	31.9	....
	Interior	....	28.5	30.3	....	30.5	28.0	....	32.5	32.7	33.5	31.0	34.4
Reduction of area	Edge	....	56.2	....	56.5	....	....	58.0	....	....	....	56.7	....
	Interior	....	59.5	56.1	....	59.2	55.3	....	59.1	55.9	60.4	61.4	57.2

Comparison of Nickel Steel and Carbon Steel.

The varying conditions affecting the physical properties of steel seem to have no greater effect on one metal than on the other, except upon the tensile strength. The greater relative reduction in strength of nickel steel with increasing thickness of material causes the elastic ratio for the thicker metal to be higher for the nickel steel than for the carbon steel, as shown by the comparisons in Table 12.

TABLE 12.—Elastic Ratio.

	⅜-in. plate.	½-in. plate.	¾-in. plate.	1-in. plate.	Tested at:	Speed, 1 in. in:	Location.
Nickel steel	60.1	58.0	57.8	*54.5	Drexel.	6 min.	Edge.
Carbon steel	*60.8	57.8	53.7	*45.0	"	6 min.	Edge.
Nickel steel	60.0	56.3	62.6	57.0	Lukens	3 min.	Interior.
" "	58.1	55.9	56.1	.....	"	10 sec.	"
Carbon steel	*64.4	58.5	54.8	*49.0	Drexel.	6 min.	Edge.
" "	60.7	55.2	51.1	48.7	Lukens	3 min.	Interior.
" "	58.6	51.4	48.7	46.1	"	10 sec.	"

* Speed, 1 in. in 2 min.

Nickel steel is not as ductile as carbon steel, yet it is not brittle in any sense. The yield point, or, as it is usually called, the elastic limit, may be safely taken as a minimum at 60 000 lb., as compared with the 30 000 lb. usually specified for carbon steel. In tests of the 1-in. material, a lower value was obtained, but the speed was slower

than in mill practice; and the carbon steel, tested similarly, gave also a lower value than 30 000 lb.

The ultimate strength varies with the thickness, speed, and the location of the specimen, but, as compared with the results of carbon steel, a value between 100 000 and 120 000 lb. may be obtained, and even 110 000 lb. as a minimum, if the conditions mentioned are not restricted—and it is not usual to restrict them.

An elongation of 15% in 8 in., and a reduction of area of 40% may be safely taken as the minimum. In almost every instance the fractures were silky, uniform, and free from laminations, pipe, etc. The only variation was in four of the plate tests and in the $\frac{3}{4}$-in. angle, these fractures being partly fine crystalline or granular.

The usual high elastic ratio for nickel steel is not found in these tests, probably because the speed of machine was much slower than is generally used, and the determination of the yield point was more accurate. As already stated, the methods in vogue at most mills would give a very high but inaccurate value for the yield point or elastic limit of nickel steel.

The tests were made at Drexel in February and March, at Pencoyd in March, and at Lukens in April, 1906.

Tensile Tests of Punched, Reamed, and Punched-Riveted Specimens.

These tensile tests were arranged, as far as possible, to show relatively what changes are effected in the physical properties of nickel steel and carbon steel by the shop operations of punching holes full size, of sub-punching and reaming to full size, and of riveting punched work. The full-sized holes were $\frac{15}{16}$ in. in diameter, the sub-punched holes, $\frac{11}{16}$ in. in diameter; $\frac{1}{8}$ in. of material, therefore, was reamed away.

It is hardly necessary to repeat here that the usually accepted opinions are that shearing the metal, as in punching, forms numerous incipient cracks radiating from the edge, and also that the material around the hole is hardened by the pressure necessary to force the punching out. It is believed that if these cracks and this hardened ring are cut away, all injury done to the material is removed. The opinions as to the effect of riveting are, perhaps, not so positive, because the subject has not been investigated experimentally; it was only introduced into this series after other tests had been made. The object was to determine the annealing effect, if any, on the material due to the driving of the hot rivet.

Test specimens, 3 in. wide and 18 in. long, were cut from the $\frac{3}{8}$-in. and the $\frac{3}{4}$-in. plates. The "punched" and the "reamed" pieces were laid out side by side in every instance, in order to give results as directly comparable as possible; the "punched-riveted" pieces were cut from available surplus material. Through an error, one piece of

carbon steel, which it was intended to punch full size, was sub-punched and reamed, and *vice versa*. The edges of all holes and plates were rounded slightly by filing to remove burrs.

Pieces *TPPL* 13 and 33, *TAPPL* 46, 140, and 142 were cut from the edge of the plate, but this does not seem to have influenced the results as in the tensile tests of plain specimens, probably because of the greater width of 3 in. The punching, reaming, and riveting were done in the bridge shop at Pencoyd in the customary way. The diameter of the die used in the punching was $\frac{1}{16}$ in. larger than that of the punch. To prevent injury to the material in cutting off the heads of rivets, the latter were driven with a plate, 3 by $\frac{1}{16}$ by 9 in., on each side of the specimen. These plates served another purpose, not intended, namely, the lessening of the amount of compression in the material under the head of the rivet, caused by the pressure of the cup of the riveting machine. The pressure on the rivet in driving was probably about 30 tons per sq. in., and the annealing effect, if any, was entirely destroyed by the compression around the hole. The metal under the head was bright, and the calipering showed a reduction of thickness averaging as follows:

	Carbon steel.	Nickel steel.
$\frac{3}{8}$	0.003 in.	0.003 in.
$\frac{3}{4}$	0.007 "	0.003 "

All the testing was done on the testing machine at Drexel Institute, and the autographic recorder was used to plot the curve of each test; and from this record the elastic limit, yield point, and ultimate strength were determined.

Yield Point.—As in the tensile tests of plain specimens, at about the yield point, the load was removed from several specimens and the permanent set carefully measured by dividers and checked with the card record, the results shown in Table 13 being obtained:

TABLE 13.—Beam Readings.

Mark.	Yield point, in pounds.	Load released at: Pounds.	Permanent set in 6 in., by dividers: Inches.
CTAPPL 46	29 500	21 000	None.
		30 200	Widened line.
" 140	66 500	66 000	None.
		68 000	Widened line.
		72 000	0.05
" 142	68 000	70 000	0.02
TAPPL 46	62 000	46 700	None.
		63 000	None.
		73 000	0.03
" 140	128 000	112 000	Widened line.
" 142	120 000	120 000	0.01
Heat No. 16 080:			
TPPL 3 8	53 200	54 000	0.01 scant.
TPPL 3 4	100 000	96 000	Widened line.
		104 600	0.02 scant.
TPRL 3/8	52 000	52 000	Widened line.
		56 000	0.02 scant.
TPRL 3/4	98 800	90 000	Widened line.
		102 800	0.015

The yield point, as before, was obtained from the curve at the intersection of a line 0.01 in. away from, and parallel to, the straight portion of the record. While there is a fair agreement between the results obtained by the two methods described, it is not as close as in the tensile tests on plain specimens, and an examination of the curves will show the reason. The deflection from a straight line is so gradual and, in those of the nickel steel, so slight, that it is impossible to obtain, by any practical means, the exact point where a permanent set first takes place. Yet the loads recorded in these tables are closely approximate (with two or three exceptions mentioned later), so that, for comparative purposes, the data obtained are of sufficient value.

In all the curves of the carbon steel there is a decided change of direction, corresponding to the stretch occurring at the yield point for the plain specimens. In these tests this change varies in form and position. In the tests of the $\frac{5}{8}$-in. material it occurs just before the maximum strength is developed; and, in those of the $\frac{3}{4}$-in. material, it is found close to the yield point. There is a sufficiently marked change in the curve at what has been taken as the yield point to substantiate the tabulated figures. In the curves for tests from Heat No. 16 080, similar changes occur, but not in the other nickel-steel tests —the former material is a softer grade of steel. The $\frac{3}{4}$-in. material is for all steels softer than the $\frac{5}{8}$-in. material; the variations described may be assumed, therefore, as being conditional upon the grade of steel.

All these tests were made in a similar manner, a speed of 1 in. in 20 min. being used just before and until the yield point was well passed, and a speed of 1 in. in 6 min. at the beginning and at the instant of breaking.

Tables 34, 35, and 36 show the results of each test in detail.

1. Curve *CTPRL* No. 7 differs from the other three in that it has a higher yield point and is straighter within this point. The autographic attachment did not work properly.
2. Curve *TPRL* No. 7 is slightly straighter within the yield point than the other three curves, but it is not abnormal.
3. Curve *TPPL* No. 111 differs for the same reason—the curve is flatter.
4. Curves for *CTAPPL* are all irregular; the autographic attachment did not work properly in describing any of these curves, but the yield point is well marked, so that the results are fairly accurate.
5. Curve *TAPPL* No. 141 differs from the others for the same reason. These curves are all so flat that it is practically impossible to obtain the yield point accurately.

The other curves are all normal.

Examination of Results.—Tables 34, 35, and 36.

In the comparisons in Tables 14 and 15 it must be borne in mind that these tests were made on specimens 3 in. wide, while the plain specimens were but 1½ in. wide. It is well known, also, that short grooved specimens give a higher ultimate strength than the usual 18-in. specimen. It is not probable, therefore, that all the effects described are the direct result of alterations in the structure of the material about the hole.

TABLE 14.—Percentages Based on Results of Tests of Reamed Specimens.

Steel.	Physical property.	⅞-in. Material.			¾-in. Material.		
		Reamed.	Punched.	Punched-riveted.	Reamed.	Punched.	Punched-riveted.
Nickel..	Yield point	95	102	113	114	111	130
Carbon.	" "	102	105	107	112	124	128
Nickel..	Ultimate strength	101	91	94	104	89	82
Carbon.	" "	103	93	93	107	94	81
Nickel..	Elastic ratio	94	112	121	110	125	157
Carbon.	" "	100	113	115	104	132	158

TABLE 15.—Percentages Based on Results of Tests of Plain Specimens.

Steel.	Physical property.	⅞-in. Material.		¾-in. Material.	
		Punched.	Punched-riveted.	Punched.	Punched-riveted.
Nickel	Yield point	108	120	97	113
Carbon	" "	103	105	111	114
Nickel	Ultimate strength	91	93	85	79
Carbon	" "	91	91	88	75
Nickel	Elastic ratio	120	130	114	143
Carbon	" "	114	115	126	152
Nickel	Elongation in 4 in.	62	91	50	46
Carbon	" "	77	80	39	14
Nickel	Reduction of area	51	57	39	22
Carbon	" " "	75	64	28	3

Punching, sub-punching-and-reaming, and riveting, in every instance, alter the ¾-in. material more than they do the ⅞-in. material, in both nickel and carbon steels.

Punching $\frac{15}{16}$*-Inch Hole.*—Punching raises the yield point and lowers the ultimate strength. The yield point of the nickel steel is affected to a smaller extent than that of the carbon steel, and the ultimate strength more, but the difference is not great.

Reaming—From $\frac{11}{16}$*-Inch to* $\frac{15}{16}$*-Inch.*—The process of sub-punching-and-reaming raises both the yield point (with but one exception,

namely, that of the ⅜-in. nickel steel) and the ultimate strength of both steels, and in almost the same proportion. The effect is less than that of punching full size, probably because the punched hole is smaller, and the injury, therefore, less, and because the reaming removes the greater part of the affected area. Nickel steel, on the whole, is affected to a smaller extent than carbon steel, but the difference is not uniform, and is small. The elongation and reduction of area cannot be compared directly with those of the plain specimens because of the marked difference in the shape and character of the specimen. Comparing the results of punching with those of sub-punching-and-reaming, however, shows that the ductility of the material is injuriously affected by punching without subsequent reaming. The difference between the effects of punching and reaming upon the ⅜-in. nickel steel is greater than it is upon the corresponding carbon steel, but it is less upon the ¾-in. material.

Riveting.—Riveting raises still further the yield point, and lowers the ultimate strength. The elongation and reduction of area are also smaller than for the reamed specimens. Upon the ⅜-in. material, however, riveting seems to be beneficial, but upon the ¾-in. material it adds to the injury resulting from punching. Nickel steel is affected much less by riveting than carbon steel, probably because its resistance to compression is greater.

The ¾-in. material being a softer steel than the ⅜-in., it was to be expected that the effect of riveting upon it would be greater.

In comparison with the carbon steel, the nickel steel in these special tests shows up very favorably, on the whole.

TABLE 16.

Kind of specimen.	⅜-IN. MATERIAL. AREA = 1.125 SQ. IN.			¾-IN. MATERIAL. AREA = 2.25 SQ. IN.		
	Actual yield point.	Actual ultimate strength.	Elastic ratio.	Actual yield point.	Actual ultimate strength.	Elastic ratio.
NICKEL STEEL.						
Plain specimens	76 000	128 000	59	140 000	243 000	58
Reamed "	51 000	91 000	56	110 000	174 000	63
Punched "	55 000	82 000	67	106 000	147 000	72
Punched-riveted specimens.	61 000	85 000	72	124 000	137 000	90
CARBON STEEL.						
Plain specimens	42 000	70 000	60	72 000	137 000	53
Reamed "	29 000	48 000	60	57 000	102 000	56
Punched "	30 000	43 000	70	64 000	90 000	71
Punched-riveted specimens.	30 000	43 000	70	67 000	78 000	86

The comparisons (in Table 16) of the actual yield point and ultimate strength of these specimens with those of the plain specimens, on the basis of a sectional area of 3 by $\frac{3}{8}$-in. and 3 by $\frac{3}{4}$-in. show that the loss of strength is less for the nickel steel in $\frac{3}{8}$-in. material than for the corresponding carbon steel, and greater for the $\frac{3}{4}$-in. material. The differences are small.

TABLE 17.—Percentage of Loss, Based on Plain Specimens.

Steel.	Physical Property.	$\frac{3}{8}$ in. Material.			$\frac{3}{4}$-in. Material.		
		Reamed.	Punched.	Punched-riveted.	Reamed.	Punched.	Punched-riveted.
Nickel	Yield point	33	28	20	21	24	11
Carbon	"	31	39	39	21	11	7
Nickel	Ultimate strength	29	36	34	28	40	44
Carbon	"	31	39	39	26	34	43

TABLE 18.—Comparison of Assumed Yield Point with Elastic Limit.

(In Pounds per Square Inch.)

Material.	Physical property.	Nickel Steel.				Carbon Steel.			
		Plain.	Reamed.	Punched.	Punched-riveted.	Plain.	Reamed.	Punched.	Punched-riveted.
$\frac{3}{8}$-in.	Elastic limit	60 000	49 000	50 000	55 000	35 000	28 000	30 000	27 000
	Yield point	67 000	64 000	69 000	77 000	39 000	40 000	41 000	41 000
$\frac{3}{4}$-in.	Elastic limit	53 000	53 000	51 000	61 000[c]	29 000	26 000	32 000[a]	35 000[b]
	Yield point	62 000	71 000	69 000	80 000	33 000	37 000	42 000	43 000

a. No. 126 not included.
b. Pencil of autographic recorder did not work properly; the figures, therefore, may not be accurate.
c. Curves are so flat that the figure may not be accurate.

Table 18 shows that within the yield point there are changes in the material, as shown in the autographic curves. The departure of the record from a straight line is what is meant by elastic limit.

The number of tests was too small, and the apparatus used was not sufficiently delicate to furnish much more than a suggestion that these operations, necessary to the fabrication of bridge members, weaken the perfect elasticity of the material. The ductility is also lessened. The method used for determining the yield point in these tests does not give such uniform or accurate results as in the tests of plain specimens. In the specimens, the ratio of stretch to the load increases very

gradually in these tests, especially in the case of nickel steel, the records of which are almost straight.

The exceptional results shown for the $\frac{3}{4}$-in. material may have been caused by the bending of the test pieces during punching and riveting. This treatment would have a tendency to raise the recorded value for the elastic limit, though actually it would be less. The results for the punched-riveted specimens of $\frac{3}{4}$-in. material may be inaccurate as noted.

The fractures are interesting. The $\frac{3}{8}$-in. material, on the whole, was silky, though the nickel steel in the punched and riveted specimens was partly crystalline.

The $\frac{3}{4}$-in. material of nickel steel was fine crystalline, as was also that of carbon steel, with the exception of the reamed specimens. The nickel steel, however, was much finer-grained, and showed more plainly lines like magnetic lines radiating from the hole. Under the skin the fracture was silky, and this was more marked in the nickel steel. The material near the hole was also silky, but this was more marked in the carbon steel.

The break occurred, with two or three exceptions, simultaneously on both sides. The difference in any case was only a fraction of a second. Those with a crystalline fracture were always unexpected, with no drawing down of the breaking section.

The tests of the punched and of the reamed specimens were made in March, and those of the riveted specimens in June, 1906.

Bending Tests, Plain Specimens.

Next to the tensile test, the test most frequently made is the bending of a specimen piece of steel until it is proved that the ductility of the material is sufficient to satisfy the specifications. This test is also an indication of the absence of high phosphorus.

The bending may be done in various ways: On a U-shaped block, under a steam hammer, by being pushed through an opening in an anvil by a plunger, or, in the case of iron and soft steel, by being wrapped around a mandrel, one end of the specimen being held rigidly and the power being applied at the other end by an eccentric cam.

This last method makes the most perfect bend, but for heavy sections of medium-carbon steel and especially of nickel steel, it is not practicable. The method to be followed is not often specified, yet it is an important factor, as is the speed of machine in the tensile test or the taper of the drift-pin in the drifting test.

These tests were made in the testing room of the Lukens Iron and Steel Company through the kindness of Mr. C. L. Huston, Vice-President, and Mr. Howard Taggart, Engineer of Tests.

The specimen was laid on an anvil over a rectangular opening, wider at the top than at the bottom. The piece was bent by being

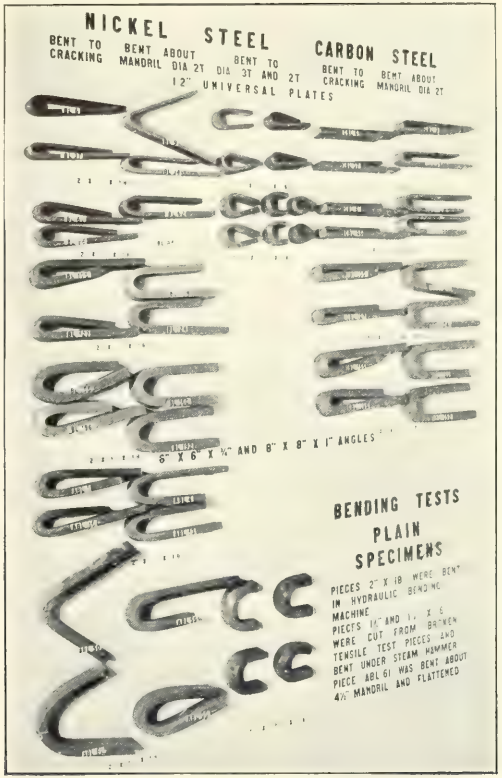

NICKEL STEEL
BENT TO CRACKING
BENT ABOUT MANDRIL DIA 2T
BENT TO DIA 3T AND 2T
CARBON STEEL
BENT TO CRACKING
BENT ABOUT MANDRIL DIA 2T
12" UNIVERSAL PLATES
6" X 6" X ½" AND 8" X 8" X 1" ANGLES
BENDING TESTS
PLAIN
SPECIMENS
PIECES 2" X 18" WERE BENT IN HYDRAULIC BENDING MACHINE
PIECES 1½" AND 1" X 6" WERE CUT FROM BROKEN TENSILE TEST PIECES AND BENT UNDER STEAM HAMMER
PIECE ABL 61 WAS BENT ABOUT 4½" MANDRIL AND FLATTENED

FIG. 1.—BENDING TESTS.

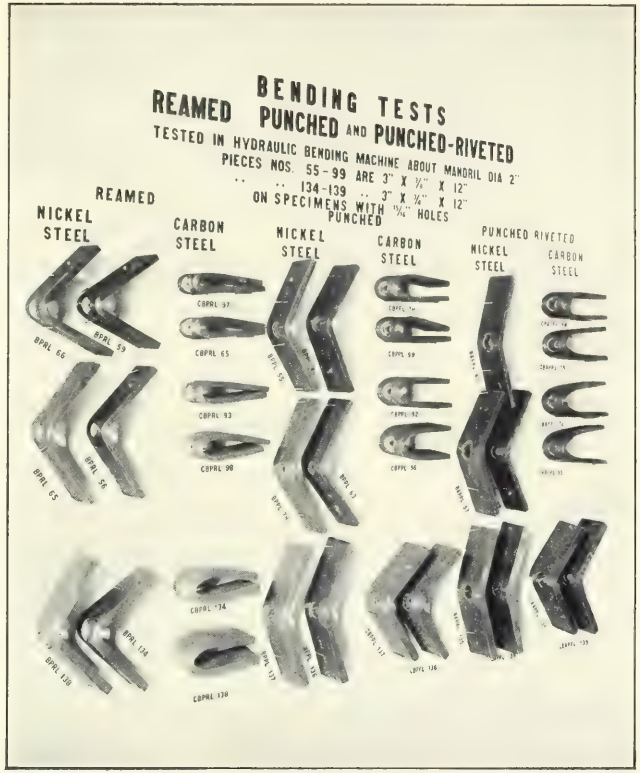


FIG. 2. BENDING TESTS.

forced through by a plunger with its end rounded to any desired radius. The opening may be made of any desired width within certain limits. The plunger is attached to the piston of a hydraulic cylinder, consequently the pressure is very uniform. The piece is only partly bent, however, and is then flattened by the same machine.

For large, thick pieces a plunger $4\frac{1}{2}$ in. in diameter is used, but ordinary structural material, up to 1 in. in thickness, is bent about a 2-in. plunger or mandrel.

The specimens for these tests were all 2 in. wide by 18 in. long, two being cut from the end of each plate, one edge of each specimen being the mill edge, finished in a universal mill, the other edge being planed. Through the carelessness of the machinist, several of the pieces were planed on both edges. One piece of nickel steel, *BL* 186, cut from the 1-in. plate, was lost, and a piece cut from the interior, marked *BL* 196, was substituted. All edges were smoothed by filing to remove burrs.

One-half of the total number of test pieces were first bent by the 2-in. mandrel, and afterward flattened until they cracked, or to 180° flat. This was done in order to find out how much bending the material would stand. All the carbon-steel specimens bent 180° flat, or until the capacity of the machine was reached, with a slight crack appearing in one piece only. The nickel steel also bent 180°, but cracked before the inner surfaces came in contact. With one exception, the nickel steel bent with a maximum radius of bend, such as would be formed about a mandrel having a diameter equal to 1.7 times the thickness of the material. One piece of angle material, *ABL* 61, was bent about the $4\frac{1}{2}$-in. mandrel, but the bend was not so good.

The results are shown by the photograph, Fig. 1, Plate CII.

It seemed from these tests that the material would stand bending about a mandrel of a diameter equal to twice the thickness of the material without cracking; hence four of these mandrels were made, one for each thickness of material. If the material had bent to the diameters intended, probably the results would have been as expected, but the bending was too localized and an acute angle was formed, so that in no instance was the radius of the bend the same as that of the end of the mandrel. Cracks, therefore, were developed, the amount of bending being practically the same as before. One piece, *BL* 85, from the $\frac{1}{2}$-in. plate, broke, and one-half was evidently thrown on the scrap pile and destroyed. The $\frac{3}{4}$-in. and 1-in. material did not show any cracks. In order to prove positively that the $\frac{3}{8}$ and $\frac{1}{2}$-in. plate material and the 1-in. angle material would stand a bending of 180° without cracking, with a diameter of inner surface of bend of $2t$ (t = thickness of material), the broken ends of some tensile test pieces were tried. This is considered a severe test, because of the straining, beyond the elastic limit, of all parts of the piece in tension. Many of the pieces were badly cut by the wedges.

The bending was done at Pencoyd under a small steam hammer. The load, however, was not applied to one place continuously, as in the Lukens tests, but was distributed over the middle of the piece, causing a gradually increasing curvature until the desired radius was obtained. One-half of the test pieces were bent to a diameter equal to three times the thickness and one-half to that of twice the thickness. Two pieces of $\frac{1}{2}$-in. material broke because too sharp an angle was formed. These bends were very satisfactory, and proved that, by careful manipulation, almost any desired degree of bending could be obtained without fracture.

These results are shown in the photograph, Fig. 1, Plate CII.

It was not expected that the nickel steel would stand the same bending as the carbon steel, but it was believed that it would stand a sufficient amount to prove its toughness or lack of brittleness.

Wherever in the tables, in columns headed "Angle of bend," a number in parentheses appears, the unenclosed number is the angle to which the material bent when the crack first developed.

The angle of bend and the radius of the bend were obtained from sketches made by tracing carefully the outlines of the bent specimens. The radius was found by deducting, from the radius of the outside surface of the bend, the thickness of the specimen. The radius of the outside surface could be determined easily and with fair accuracy by being taken in a plane midway between the edges. Cracks usually developed first in this portion and spread to the edges. The testing was done in March and June, 1906.

Bending Tests, Punched, Reamed, and Punched-Riveted Specimens.

The bending tests on nickel and carbon steels, like the corresponding tensile tests, were arranged to show relatively the effect of punching material full size, of sub-punching and reaming to full size, and of riveting punched work. The full-sized holes were $\frac{15}{16}$ in. in diameter and the sub-punched holes, $\frac{11}{16}$ in. in diameter, $\frac{1}{8}$ in. of material, therefore, being reamed away.

The test was not included in the original scheme of testing, hence the specimens had to be cut from the excess material. None of the $\frac{3}{8}$-in. plate of nickel steel was available, and only small pieces of the $\frac{1}{2}$ and $\frac{3}{4}$-in. plates. For nine of the test pieces of nickel steel, it was necessary to use the pieces 4 by $\frac{1}{2}$ by 12 in., designed and used for testing rivets in single shear. The greatest load on any joint was 63 400 lb., or 31 700 lb. per sq. in. This did not exceed the elastic limit, which from Table 30 was 62 000 lb. An examination of the results in Table 37 does not furnish any evidence that the previous use had worked an injury to the material. The location of the specimens in the plates seems to have had no influence on the results.

The specimens were machined to a width of 3 in. with these exceptions: Pieces *BPRL* 65, *BPPL* 78, *CBPRL* 65, *CBPPL* 78, and *CBAPPL* 75 were $2\frac{15}{16}$ in. wide. The punching, reaming, and riveting were conducted by the usual shop methods. The diameter of the die used in punching was $\frac{1}{16}$ in. larger than that of the punch. The same riveter as for the tensile tests, and the same scheme for protecting the specimens by a $\frac{1}{16}$-in. plate on each side, were used. The metal under the head of the rivet was bright, and, no doubt, was compressed as in the tensile-test specimens. As comparative results only were sought, there was no experimenting with mandrels of different diameters. The pieces were all bent, as described under the tests of plain specimens, in the testing room of the Lukens Iron and Steel Company, with a mandrel rounded on the end to a diameter of 2 in., a continuous pressure being slowly applied at the middle over the hole.

The edges of pieces and of holes were smoothed by filing. This was overlooked for the first three pieces tested, namely, *CBPRL* 65, 93, and 98; consequently, two pieces of nickel steel, *BPRL* 56 and 65, were tested in the same condition for comparison. The filing does not seem to have aided the carbon steel, but its effect on the nickel steel was marked. The two filed pieces bent 104° and 108°, respectively, and the unfiled pieces, 75° and 77°, with a corresponding difference in the radius of bend.

The results are given in Tables 37 and 38, and in the photograph, Fig. 2, Plate CII.

In every instance punching, sub-punching and reaming, and riveting, alter the $\frac{3}{4}$-in. material more than the $\frac{3}{8}$-in. material of both nickel steel and carbon steel. The $\frac{3}{4}$-in. plain specimens bent without cracking through the greater angle; but, in these tests, the reverse was true. The difference between the effects of punching and of reaming upon the $\frac{3}{8}$-in. nickel steel is greater than it is upon the carbon steel, but, upon the $\frac{3}{4}$-in. material, it is less.

Riveting increases the effect of the punching on both thicknesses of both steels and in about the same proportion. As a whole, these conclusions are the same as those drawn from the tensile tests. It could not be expected that nickel steel would bend through as great an angle as carbon steel. In the photograph, Fig. 2, Plate CII, the bent specimens are arranged side by side for comparison.

These tests were made in March and June, 1906.

Nicked Bending Test.

The following pieces, *NBL* 27, $2\frac{1}{2}$ by $\frac{3}{8}$ by 8 in, *NBL* 120, 2 by $\frac{3}{4}$ by 18 in., and *NBL* 189, 1 by 1 by 18 in., of both steels, were nicked $\frac{1}{8}$ in. and fractured in two places by bending. The fractures were uniform in appearance and free from laminations, pipe, or other defects. The fractures of the nickel steel were finer-grained than those of the carbon steel.

This determination of character of fracture was the sole object of the test.

Drifting Tests.

The consideration given this test, when the lay-out of tests was being prepared, was long and thorough. Specifications, for steel superstructures of all the large railroads, and of consulting engineers, were examined and tabulated. The specification of the drifting test seemed to be a purely arbitrary matter, as the amount of drifting a $\frac{13}{16}$-in. or a $\frac{15}{16}$-in. hole, punched full size and spaced $1\frac{1}{2}$ in. from a sheared or rolled edge, was required to stand without the edge of the plate or the periphery of the hole cracking, ran from 25 to 33%, 38%, 50%, and, in four instances, to 100 per cent. The other standard requirements of carbon steel for bridges are given later.

In the arrangement of holes finally decided upon, almost every recent specification, even the most severe, has been included. This explains the irregular grouping of holes. The number was limited by the size of the coupon selected, namely, 12 by 12 in.

With but two exceptions the hole required to be drifted was a punched hole—these two specified a $\frac{13}{16}$-in. hole reamed from a diameter of $\frac{11}{16}$ in. Holes of this sort were included, also holes drifted from the solid. The $\frac{3}{8}$-in., $\frac{3}{4}$-in., and 1-in. plates were selected, the first two because of the great bulk of structural material represented thereby, and the 1-in. plate, because it was proposed to use nickel-steel plates of this thickness in a large bridge, plans for which were being prepared.

Only one specification was found in which a taper for the drift-pin was specified, yet this is as important as the size and location of the hole. A pin with a large taper bulges the metal around the hole, and the edge cracks because of this, and not because the limit of enlargement has been reached. The ideal pin is one with a scarcely perceptible taper. A taper of 1 in 12 was adopted for this test, and several pins of this kind were made of specially hardened blue-chip and Sanderson steel.

The preparation of the test specimens and the testing were done at the Pencoyd Iron Works, the drifting being done by driving the pins with a heavy steam hammer, first through one side and then through the other, about $\frac{1}{2}$ in. at a time. In this way excessive bulging was prevented. It will be noted that the largest cracks and the greatest number of them are always on the side where the bulging is the greater. The holes were all carefully laid out, but, in punching, a slight shifting of some took place; the amount is not large enough to be of importance. Burrs were removed from the edges of the holes, but no filing was done. The diameter of the die used was $\frac{1}{16}$ in. larger than that of the punch.

Tables 39, 40, and 41, together with the photograph, Plate CIII, show the effect of drifting on specimens drifted according

DRIFTING TESTS

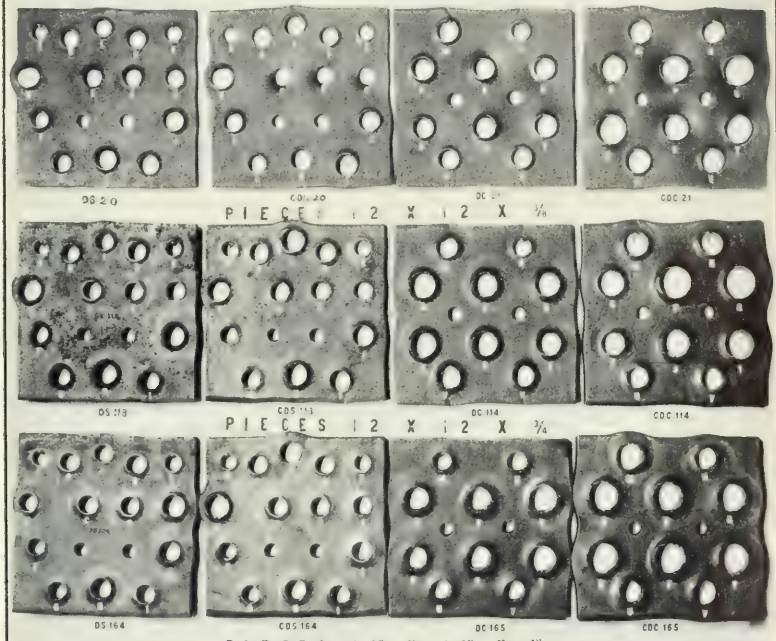
ON SPECIMENS SHEARED FROM 12 UNIVERSAL PLATES

DRIFTED ACCORDING TO SPECIFICATIONS

NICKEL STEEL — CARBON STEEL

DRIFTED TO CRACKING

NICKEL STEEL — CARBON STEEL



PIECES 12" X 12" X 1"

HORIZONTAL EDGES SHEARED
VERTICAL ROLLED

HOLES A E F G K L	IN PIECES NOS.	20 AND	113	PUNCHED	13/16	
B C D H I J		20	113	"	15/16	
M P Q R V		21	114	"	15/16	
O U		21	114	"	11/16	REAMED TO 15/16
N S T		21	114	DRILLED	15/16	FROM SOLID
A TO V		164	165	"	15/16	" "

DRIFT PINS TAPERED 1 IN 12

to the requirements of the specifications mentioned, and Tables 42, 43, and 44 and the same photograph show the effect of carrying the drifting until cracks were developed at the edge of plate or the periphery of the hole. The nickel steel drifted with greater difficulty than the carbon steel. Through an oversight, Hole *K*, in *BS* 164, was not drifted as much as intended.

An examination of the tables shows that the carbon steel drifted better than the nickel steel. The difference is not marked in the tests made according to specifications; but in the tests to cracking, comparing the enlarged diameters of the holes, the $\frac{3}{8}$-in. nickel-steel plates drifted 58% as well, the $\frac{3}{4}$-in. material, 72% as well, and the 1-in. material, 67% as well as the carbon-steel plates, but the carbon steel is cracked to a larger extent than the nickel steel. It is probably fair to conclude that nickel steel will stand about 70% as much drifting as carbon steel, and satisfy the drifting requirements of many specifications written for medium steel. A hole spaced, as in the usual practice, from a sheared edge, will not stand drifting to the same extent as one spaced near a rolled edge. These tests are too few to give any conclusive figures, but probably the ratio is nearly as 1 to 2. Punched holes seem to drift as well as, if not better than, either reamed or drilled holes. The percentages giving extent of enlargement (Tables 42 and 43) may be arranged to show this as follows:

TABLE 19.—Percentage by Which Holes Were Enlarged.

Hole edge.	Material.	Punched.		Reamed.		Drilled.	
		Sheared.	Rolled.	Sheared.	Rolled.	Sheared.	Rolled.
3/8-in.	Nickel steel	57 50	47 50	47	60	33	60
3/4-in.	" "	53 60	80 87	73	87	33	73
3/8-in.	Carbon steel	73 73	110 117	73	83	67	100
3/4-in.	" "	80 67	133 67	60	87	80	113

These tests were made in April and May, 1906.

Close-Punching Test.

The close-punching test is rarely specified, because it furnishes but little information about the physical properties of material not obtained by other and more usual tests. It would show up brittleness or excessive hardness. In this series its value was in causing to be punched at one time, a sufficient number of holes in the nickel steel of different thicknesses to permit forming an idea of the effect on the machine and punches, and also to show the ability of the metal to stand close punching.

A rather light punching machine in the shop at Pencoyd was used; and, while no part gave way, the $\frac{3}{4}$-in. material was too heavy for it. Twenty-eight $\frac{13}{16}$-in. holes, and twenty-five $\frac{15}{16}$-in. holes were punched in each of the two pieces, *CP* 22 (12 by 12 by $\frac{3}{8}$ in.), and *CP* 115 (12 by 12 by $\frac{3}{4}$ in.), and about the same number in the corresponding carbon-steel pieces. The arrangement of these holes and the results of the test are shown in detail on Fig. 1, Plate CIV.

Two punches were broken, one by stripping and one by shearing, while punching the nickel steel. The steel in these punches was considered exceptionally good, but the toolmaker was not familiar with it, and frequent breakages had happened in regular work, consequently, the blame cannot be laid altogether upon the nickel steel. At the time, the shop management regarded this work as more of a test for the punches than for the nickel steel. Before this, in connection with this investigation, considerable punching of nickel steel had been done at Pencoyd, with but one breakage, a $\frac{15}{16}$-in. punch. One punch and one die were broken in punching material for struts at Ambridge. Several holes, $\frac{11}{16}$ in. in diameter, were punched in $\frac{3}{4}$-in. nickel-steel plates, both in preparing test specimens and in preliminary experimental work. As regards the effect upon the punch, this is not a serious matter. The heavier punching machines now in the large shops would not be over-strained were nickel steel to be introduced, but the smaller ones unquestionably would be.

The rapidity of punching would not differ from present practice were nickel steel introduced, though, at the outset it might, because the "punching" flies out with considerable force and with a loud sharp report, and men unaccustomed to this think there is danger.

The punched hole in nickel steel is very different from that in carbon steel—its edges are clean and smooth, with no appreciable compression of the "punching." For this reason it is thought by some that punching is less injurious to nickel steel than to carbon steel. A discussion of this will be found under tensile tests and bending tests of punched specimens.

The holes in the "close-punching" coupons at the left and top edge were spaced so that $\frac{1}{8}$ in. of metal was left between the edges of adjacent holes, $\frac{3}{16}$ in. between the edge of the hole and the rolled edge, and $\frac{1}{4}$ in. between the edge of the hole and the sheared edge.

The diameter of the die used was $\frac{1}{16}$ in. larger than the punch.

The partitions between the holes actually varied from $\frac{1}{16}$ to $\frac{1}{8}$ in., and between the hole and the sheared and rolled edges from $\frac{3}{16}$ to $\frac{5}{16}$ in. The actual distances are so much smaller than stipulated in any specification ever written that a variation of $\frac{1}{16}$ in. or more is unimportant.

The photograph shows the die side of these plates.

The effect of the punching is practically the same for both steels. Where the partitions between the holes in the nickel-steel plates are

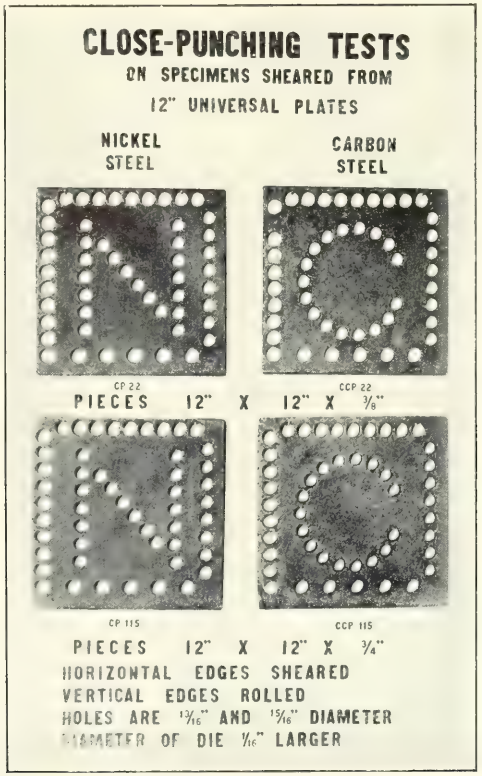


FIG. 1. CLOSE-PUNCHING TESTS.

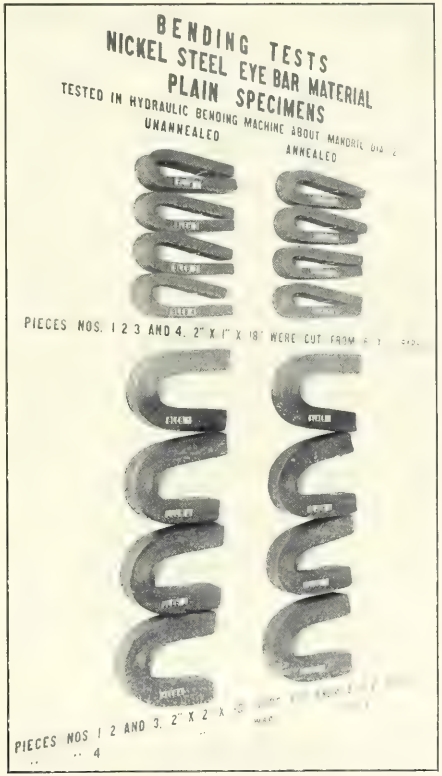

BENDING TESTS
NICKEL STEEL EYE BAR MATERIAL
PLAIN SPECIMENS
TESTED IN HYDRAULIC BENDING MACHINE ABOUT MANDRIL
UNANNEALED
ANNEALED
PIECES NOS. 1 2 3 AND 4. 2" X 1" X
PIECES NOS. 1 2 AND 3. 2" X 2"
" " 4

FIG. 2. EYE-BAR MATERIAL TESTS.

shown broken down, this resulted from inaccurate punching; as explained, the operator was nervous. The deformation of each hole by the succeeding one is alike in both steels, it occurs on the die side only, all holes on the punch side being truly circular in form; there is, however, a slight depression of the walls between adjacent holes. There was no deformation of the metal between the other holes in these plates. This test was made in March, 1906.

Hammering-Flat Tests.

A clause covering the hammering-flat test can be found in one or two specifications. It is strictly a punishment test to detect brittleness. Material is rarely treated in this way in bridge-shop practice except in fillers; but, in riveted-steel pipe manufacture, the corners of many plates must be scarfed by cold hammering.

The test was made on 2 by 8-in. pieces, marked *HL* 14 and *HL* 131, cut from the $\frac{3}{8}$-in. and the $\frac{1}{4}$-in. plates, respectively, of each steel, under a light steam hammer. A small round was first used to localize the distortion, but only for a few blows, most of the work being done by hammering the end held on the anvil. There was very little difference between the nickel steel and the carbon steel; the latter broadened out more, but the thickness at the edge was not less and the cracks were a little larger. Nickel steel is so stiff under treatment of this kind that the strain on the hammer is very great. The results are shown in Table 20.

TABLE 20.—Hammering-Flat Tests.

Material.	Original.		Final.		Remarks.
	Width, in inches.	Thickness, in inches.	Width, in inches.	Thickness, in inches.	
Nickel steel.....	2	$\frac{3}{8}$	$2\frac{1}{16}$	$\frac{1}{32}$	No cracks developed.
" "	2	$\frac{3}{4}$	$2\frac{1}{16}$	$\frac{3}{64}$	Two small cracks started.
Carbon steel.....	2	$\frac{3}{8}$	$2\frac{1}{16}$	$\frac{1}{32}$	Two small cracks started.
" "	2	$\frac{3}{4}$	$2\frac{1}{16}$	$\frac{1}{32}$	One quite large crack.

This test was made in the forge shop, at Pencoyd, in March, 1906.

Shop-Tooling Tests.

By shop-tooling tests is meant submitting the steel to all the shop operations necessary to its fabrication, and watching closely the results.

Sawing.—It has been stated that the 8 by 8 by 1-in. angle was sawed across. A time test was not made. The saw was not injured. Probably it revolved at the usual rate, but was fed more slowly.

Shearing.—All the usual sizes of plates may be safely sheared in any of the large shops—in these tests the 12 by 1-in. and a 36 by $\frac{7}{8}$-in. were the heaviest sections sheared. The strain on the shears is considerable, and probably, were nickel steel introduced, heavier machines

would have to be built. The same time is required, whether nickel steel or carbon steel is sheared.

Punching.—A discussion of punching was introduced under the close-punching test. No difficulties greater than getting the operators accustomed to the material are presented. There would be a slight increase at the outset in the breakages of punches, but, without doubt, the toolmakers would meet this extra demand upon them.

Reaming.—Progressive shops, to-day, are adopting dry reaming with high-speed, steel tools, the only lubricant used being a graphite paste. Many shops, however, still retain the slow-speed, ordinary, steel drill with an excessive use of a lubricant, principally water. This sort of tool will not stand up with nickel steel: it over-heats, the edge crumbles, the tool binds, and the progress is painfully slow. With the special high-speed, steel tool revolving at great rapidity, nickel steel may be reamed at the same rate as carbon steel; graphite is used more liberally, and the tool wears out more frequently. In the fabrication of the struts for compression tests, nickel steel and carbon steel were being run through a drill press, side by side. The behavior of the tools was carefully watched, though no time test was made. Some of the operators, Hungarians and Poles, had trouble with the sticking of the tool, but not after they became accustomed to the material.

Drilling.—A number of holes were drilled in specimens for drifting tests, riveted-joint tests, etc., and a special time test was also made. The general observations made under reaming might be repeated here. Ordinary steel tools will not last for any length of time, but the special steels now in common use may be used successfully.

A piece cut from the 1-in. plates of nickel steel, marked *SD* 161, and one from the 1-in. plate of carbon steel, marked *CSD* 161, were used for the comparative test, with the results shown in Table 21.

TABLE 21.—Drilling Tests, with Common $\frac{15}{16}$-Inch Twist Drill.

Medium Carbon Steel.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	1	1 min. 17 sec.	144	Drill was not altered.
2	1	1 " 18 "	144	" " " "

Structural Nickel Steel.

1	$\frac{5}{8}$	1 min. 10 sec.	144	Drill was worn out.
2	$\frac{7}{8}$	1 " 26 "	144	" " " "

The ordinary drill will stand up several hours with the usual carbon steel; with nickel steel, it gets excessively hot, turns blue, the point crumbles, and the clearance wears off.

In the drilling of Hole No. 2, after 56 sec., the drill was permitted to cool off before finishing the hole. Such a tool, however, would not be used in practice.

TABLE 22.—Drilling Tests, with Cleveland High-Speed No. 10 Steel (Blue-Chip) Tool.

Medium Carbon Steel.

No. of hole.	Depth of hole, in inches.	Time consumed.	Speed, in revolutions per minute.	Remarks.
1	1	0 min. 51 sec.	210	Same drill used throughout—it was in perfect condition.
2	1	0 " 49 "	210	
3	1	0 " 49 "	210	
4	1	49 "	210	

Structural Nickel Steel.

1	1	1 min. 16 sec.	144	Same drill used in above test, after being ground, was used throughout—the point had begun to crumble at end of second hole and was in very poor condition at end of fourth hole.
2	1	1 " 16 "	144	
3	1	1 " 14 "	144	
4	1	1 " 15 "	144	

The speed was changed for the nickel steel, because of the danger of breaking the drill at the higher speed. The carbon-steel chips were steel gray, and the nickel steel, blue. No lubrication was used.

The speed, therefore, with which the carbon steel was drilled was 1 in. in 35 sec., and the nickel steel, 1 in. in 52 sec. A blue-chip tool, in ordinary work, would last half a day without sharpening, whereas, if used for 5 or 6 min. on nickel steel, it is necessary to sharpen it. This factor is the more important by far, as considerable time is wasted in the re-dressing.

This test was made in the machine shop at Pencoyd, in September, 1906.

To obtain a comparison of all factors, a much longer test would be necessary; the foregoing, however, throws some light on the subject.

Planing.—For planing, the edge planer at the Pencoyd plant was used, and a blue-chip steel cutter. An attempt was first made to determine the depth of cut that could be made in the 1-in. plate material of nickel steel and of carbon steel. Coupons 4 in. wide and 18 in. long were bolted rigidly to the planer bed. With the carbon steel it was possible to make a cut of $\frac{1}{8}$ in., but the belt could not furnish sufficient power to cut this depth in the nickel-steel plate. A $\frac{1}{16}$-in. cut was made, and, after a total depth of $\frac{5}{16}$ in. had been removed from the 18-in. piece, the edge was burned off the tool.

Successive attempts proved it possible to take a maximum depth of about $\frac{3}{32}$ in. of cut in the nickel steel; the surface, however, was

very rough and torn. Eight cuts, $\frac{1}{16}$ in. in depth, were next taken from each of the 18-in. plates, and the times noted, as follows:

	Time, in seconds.								
Carbon steel	6¼	5¾	6	5¾	6	5¾	5½	5¾	Average 5⅞
Nickel steel	6	6	5¾	6	6	6¼	5¾	5½	Average 5⅞

The carbon steel apparently had no effect on the tool. The nickel steel, after eleven cuts, had burned the edge off. A cut of $\frac{1}{32}$ in. would be probably the maximum depth that would be taken from a long plate, because of the destructive effect on the tool.

It may be concluded, therefore, that nickel steel is more difficult to machine than carbon steel, and the amount of work done in a given time would be less, probably not more than half as much. The strips from the carbon steel were short in length, and steel gray in color; the strips from the nickel steel were longer, and deep blue in color. This is evidence of the greater power necessary to cut the nickel steel. This test was made in March, 1906.

Pneumatic Chipping.—Pieces 4 by ½ in. were bolted securely by two bolts to a heavy I-beam. The workman was a regular chipper; he operated a Boyer hammer, size $1\frac{1}{16}$ by 3 in.; and the chisels were of standard size. The results are shown in Table 23.

TABLE 23.—Results of Pneumatic Chipping.

	Nickel Steel.		Carbon Steel.	
Cuts.	Time required.	Length of strips.	Time required.	Length of strips.
First cut	3 min. 9 sec.	16⅜ in.	3 min. 53 sec.	16⅜ in.
Second cut	5 " 6 "	13½ "	2 " 18 "	13½ "
Third cut	3 " 15 "	12¼ "	1 " 35 "	12⅝ "
Fourth cut	4 " 3 "	11⅜ "	2 " 50 "	11⅝ "
Totals	15 min. 33 sec.	54⅛ in.	9 min. 56 sec.	54⅛ in.
The first two cuts in the nickel steel were made by a workman using a short chisel.			All cuts made with new chisels.	
The third and fourth cuts in the nickel steel were made by another man with a new chisel taken from the stock room. (Workmen do not like these chisels because of their length; they think they cannot do as much work with them as with the short ones.)			Chisel was not affected.	
Material removed=2.58 cu. in.			Material removed=3.29 cu. in.	

Material.	Time.	Length of cut.	Amount removed.
Nickel steel	15.55 min.	54.12 in.	2.58 cu. in.
Carbon steel	9.92 "	54.12 "	3.29 " "
On a Basis of 10 Min.			
Nickel steel	10 min.	34.8 in. (64%)	1.66 cu. in. (50%)
Carbon steel	10 "	54.6 " (100%)	3.32 " " (100%)

Hand Chipping.—Pieces, 4 by ½ by 18 in., were bolted in the same manner as for the pneumatic-chipping test. The workman was an expert chipper of long experience. The results are shown in Table 24.

TABLE 24.—Results of Hand Chipping.

Nickel Steel.			Carbon Steel.		
Cuts.	Time required.	Length of strips.	Cuts.	Time required.	Length of strips.
First cut...	5 min. 0 sec.	5¼ in.	First and second cut....	5 min. 0 sec.	8 in.
Workmen rested 5 min.			Workmen rested 5 min.		
Second cut.	5 min. 0 sec. 3 " 22 " 1 " 4 " 1 " 32 "	6 in. 2⅛ " 1¼ " 1¼ "		5 min. 0 sec.	7 in.
			First cut was 10½ in. long, remainder was on second cut.		
			Second and third cuts	4 min. 0 sec.	5⅛ in.
			⅝ in. was on second cut, and 4½ in. on third cut.		
Totals.....	15 min. 58 sec.	16¼ in.	Totals	14 min. 0 sec.	20⅛ in.
In chipping the second strip, the chisel was broken after 2 min. 32 sec.; chipping ⅛ in. further turned edge of chisel. After 3 min. 22 sec. of chipping, new chisel was tried; in 1 min. 4 sec., piece was broken out of chisel, and, after another 1 min. 32 sec., a piece was broken out of a second new chisel.			Chisel was not affected.		
Material removed = 0.97 cu. in.			Material removed = 1.24 cu. in.		

Material.	Time.	Length of cut.	Amount removed.
Nickel steel.....	15.97 min.	16.25 in.	0.97 cu. in.
Carbon steel.....	14.0 "	20.12 "	1.24 " "
On a Basis of 10 Min.			
Nickel steel.....	10 min.	10.2 in. (71%)	0.61 cu. in. (69%)
Carbon steel.....	10 "	14.4 " (100%)	0.89 " " (100%)

All this work was done in the shop at Pencoyd, and, though the period of testing was short, a fair comparison may be drawn. It may be assumed that the time required to chip nickel steel would be one-half longer than for the usual carbon steel.

These tests cover practically all the shop operations except milling, and, as this is similar to planing, the same conclusions may be drawn.

It is not possible from these few tests to state even roughly what the additional cost of fabrication would be, should nickel steel be introduced.

With the addition of one or two machines and a few men, the same number of pieces of nickel steel as of carbon steel could be handled in the shop, provided sections of the same size were used in the structures; but if advantage were taken of the high tensile strength of nickel steel, to cut down the size of the sections, then the output in tons would be decreased very considerably without a proportionate decrease in operating expenses. The general expenses of sales and auditing departments, of general office, drafting-room, template shop, and, in the bridge shop itself, of laying-out, assembling, riveting, painting, and dead labor would not be affected materially.

Bearing-on-Pins Test.

The bearing-on-pins test has considerable value for comparative purposes. It was designed to give the bearing values of structural nickel steel and carbon steel and of rivet nickel steel and carbon steel on a hard steel pin 1 in. in diameter. A special contrivance, illustrated by Fig. 73, was built for this purpose.

This contrivance was also designed to test rivets in double shear, which explains the presence of a ⅞-in. hole over the 1-in. hole. The steel pin was turned to a driving fit, and was supported at four points to minimize bending. The two inner bearings are of tool steel, specially hardened and spaced with just sufficient clearance to permit easy withdrawal of the specimen after testing.

The pieces tested were cut from the 12 by 1-in. Universal rolled plates of nickel steel, Heat No. 17 673, and of carbon steel, Heat No. 33 342, three from each; also three pieces from each of two similar plates rolled from a rivet grade of nickel steel and one of carbon steel. The following physical properties of the rivet material were reported by the mill:

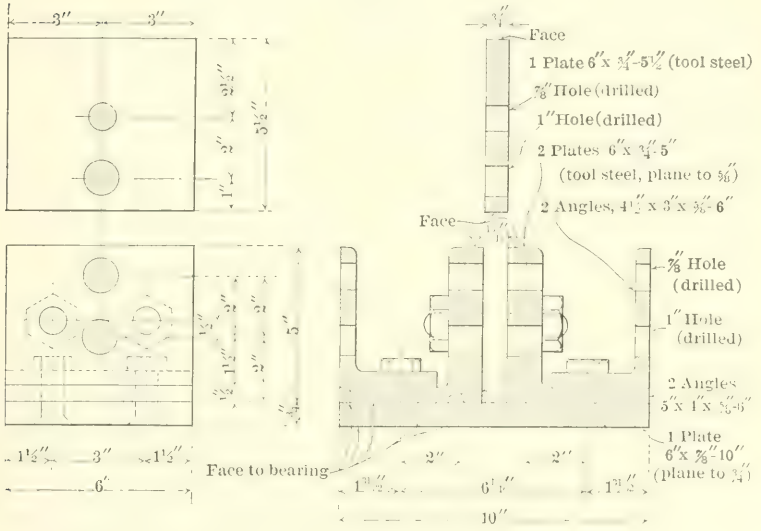
Composition.	Nickel rivet steel. Heat No. 2 096.	Carbon rivet steel. Heat No. 19 241.
Nickel	3.30	....
Carbon	0.17	0.13
Manganese	0.53	0.43
Sulphur	0.028	0.025
Phosphorus	0.012	0.028
Silicon	0.020	.....
Ultimate strength	84 200 lb. per sq. in.	54 100 lb. per sq. in.
Elastic limit	51 800 " " " "	37 600 " " " "
Elongation in 8 in.	22.5%	30.5%
Reduction of area	50.3%	59.9%

The twelve specimens were each 3.83 in. wide, 4.50 in. long, and of the thickness of the material from which they were cut. Slight variations in length were neglected, because they would not affect the read-

ings materially. The average distance from the top of the specimen to the top of the pin-hole was 3.83 in., but varied from 3.73 to 3.86 in.

All testing was done in the testing room of Tinius Olsen and Company. The load was applied by the movable head of a 100 000-lb. machine, and was read on the beam in the usual way; the compression was measured by a compressometer, reading directly to $\frac{1}{10\,000}$ in., this delicate adjustment being effected by an electric contact ringing a tap bell.* The zero readings were taken under a load of 2 000 lb., in order that all clearances in the contrivance should be eliminated from the results.

CONTRIVANCE FOR MAKING TESTS OF RIVETS IN DOUBLE SHEAR AND FOR DETERMINING BEARING VALUES OF NICKEL AND CARBON STEELS ON HARD PINS.



All rivets and bolts, ¾″ diam., drill holes, use turned bolts.
Open holes to be solid drilled after parts are finished, riveted and bolted.
Tool-steel will be furnished to shop.
Other material to be taken from stock.
Use no paint in assembling.
Face backs of angles accurately at right angles to a thickness of ½″.
Temper tool-steel after holes are drilled.

FIG. 73.

Two projecting arms rested on the base of the apparatus and two against the under side of the specimens which projected below the pin for this purpose. Under each increment of load there was a yielding of the pin and supports, as well as a compression in the specimen above the pin, and, after the yield point had been reached, a distortion of the pin-hole. This yielding varied with each test; the pin was always

* A description of this instrument may be found in Olsen and Company's Catalogue, Part C, page 16.

inserted in the same position. To determine accurately the amount of yielding of pin and supports, a piece of nickel steel with a half-hole only was used; one pair of arms, instead of resting against the specimen, rested against the under side of the pin. A number of such readings were taken before, between the specimen tests, and after. The pin, after the series of tests on the plate material was made, was found to have been bent perceptibly, hence a second pin was used for the tests of rivet material.

The half-hole piece of carbon steel was used only once, because it was so badly distorted; and the readings, therefore, were discarded.

Readings Nos. 1 and 5 were neglected, because so large a part of the deformation of pin and contrivance caused by this first loading became permanent that distorted readings would have resulted from their use. The readings are not accurate to the last figure. Readings Nos. 3 and 4 (Pin 1) and Nos. 6 and 7 (Pin 2), taken consecutively, show that the probable error through inaccuracies of readings is very small.

It will be noticed that after each loading there was an increase in the permanent deformation of the contrivance; therefore, in making the correction to the intermediate tests of specimens, this was taken into consideration. Reading No. 8 (Pin 2) shows that for some reason the conditions were changed, and that this is not the result of inaccurate micrometer observations is shown by the readings for Specimen $3N$. The conditions became again more nearly normal for $3C$.

It was impossible with the apparatus at hand to make corrections for every variable. The results of these tests are not scientifically exact, but for all practical purposes they are closely approximate. The corrected amounts of compression for each specimen were obtained by deducting, from the micrometer readings for the specimens, the micrometer readings on the half-hole pieces, Nos. 1 to 9, inclusive, in the following way: The plate carbon-steel readings are based on No. 2 (Pin 1); the nickel steel on intermediate values between No. 2, and an average of Nos. 3 and 4. The rivet steels, $1N$ and $1C$, on averages of Nos. 6 and 7; $2N$ and $2C$ on intermediate values between averages of Nos. 6, 7, and 8; $3N$ and $3C$ on intermediate values between Nos. 8 and 9.

Errors arising from this method affect only the amounts of compression and not the elastic limit, which was obtained directly from curves plotted from the original readings. The loading of the carbon steel was not carried as far as the plate nickel steel, because of the very large resulting compressions in the specimens, the curves becoming almost horizontal. The elastic limit is fairly well marked except for Curve *BRP* 116.

Tables 45 to 49, inclusive, give in detail all the readings taken and also the corrected readings.

Table 50 is arranged to show the essential features of each test for

easy comparison. The figures in parentheses in the column headed "Elastic Limit" are those obtained from the corrected readings. As shown in the column of "Original Dimensions of Pin-Hole," the pin-holes in the plate specimens were 1 in. in diameter, while those in the rivet specimens were slightly enlarged. An examination of the readings of the plate steel revealed variations for loadings under 50 000 lb. not directly traceable to the contrivance—they were greater for the nickel steel. The pin-holes in these specimens were not left perfectly smooth by the drill. In the case of the carbon steel, this surface, after a small load, became as polished as glass, but the surface of the nickel steel still showed roughnesses, even after the maximum load had been applied. The entire surface, therefore, could not have had a bearing, and as the bearing surface varied, it could be supposed that the amount of compression over the pin, under equal loads, varied also, and that the extent of this variation would decrease as the load increased. The pin-holes of the rivet steels, therefore, were scraped and smoothed with emery.

It is evident from Table 50 that the nickel steel has a very much higher bearing value than the carbon steel; both within the assumed elastic limit and beyond, the resistance to compression becoming greater relatively as the load increases. Under a load of 115 000 lb. per sq. in., the amount of compression is only one-tenth of what it is in the corresponding structural carbon steel.

These tests were made in February and March, 1906.

Bending on Pins.

Specimens 1 in. square and 18 in. long were used. The surface of the plate on two sides was left and the two others were machined. On one machine surface the pieces rested on two supports 12 in. apart—the points of bearing being rounded hard steel. The load was applied at the middle through a steel casting, with the edge rounded to a radius of $\frac{1}{4}$ in. and bolted to the movable head of a 100 000-lb. Olsen testing machine having an autographic attachment. The results were obtained from the pencil record. The movement of head within the elastic limit was at the rate of 1 in. in 12 min., and beyond this point 1 in. in 3 min. After the maximum load had been reached, the test was discontinued.

The results are given in Table 25.

The depth of the carbon-steel specimens being slightly less than those of the nickel steel, to make the results strictly comparable, the elastic limit of the carbon steel should be increased by 25 lb. and the maximum load by 50 lb.

The values for "elastic limit" in Table 25 are those obtained at the point at which the stress-strain ratio changes, as indicated by a deviation of the record from a straight line. If this point be determined as was the yield point in the tensile tests, it would be 5 100 lb. for the

nickel steel, and 2 570 lb. for the carbon steel, and for the elastic ratios, 54 and 50, and the deflections, 0.101 in. and 0.066 in., respectively.

TABLE 25.—Bending-on-Pins Tests.—Structural Nickel Steel and Carbon Steel.

Specimens, 1 by 1 by 18 in. — Supports, 12 in. between centers.

Mark.	Load per Square Inch.			Deflection at elastic limit, in inches.
	Elastic limit, in pounds.	Maximum load, in pounds.	Elastic ratio. Percentage.	
Nickel-steel, 12-in. Universal Plate—Heat No. 17 673.				
BNP 156	4 740	9 580	50	0.090
BNP 157	4 520	9 450	48	0.080
BNP 159	4 640	9 480	49	0.075
Average	4 630	9 500	49	0.082
Carbon-steel, 12-in. Universal Plate—Heat No. 33 342.				
CBNP 156	2 240	5 100	44	0.044
CBNP 157	2 460	5 220	47	0.055
CBNP 159	2 490	5 080	49	0.060
Average	2 400	5 130	47	0.053

The general conclusions would not be altered, however, the elastic limit for the nickel steel being nearly double that for the carbon steel, with a deflection only 1.55 times as great. As in the "bearing-on-pins" test, therefore, it would seem that the nickel steel was decidedly the stiffer material. The maximum load carried by the nickel steel was 1.85 times that sustained by the carbon steel.

Compression Tests of Struts.

Twelve struts were designed for this test, three of nickel steel, 10 ft. long, and three of same, 30 ft. long from center to center of end pin-holes, and the same number of each length of medium carbon steel.

Each was built of four angles, 3 by 3 by $\frac{3}{8}$ in., and two plates, 12 by $\frac{3}{8}$ in., laced on two sides with bars $2\frac{1}{2}$ in. and $\frac{3}{8}$ in., and had an area of cross-section of 17.44 sq. in. The ends were heavily reinforced to insure failure in the body of the strut.

The design of these struts is shown on Plates CV and CVI. The material was rolled by the Carnegie Steel Company, at its Homestead Works, and was fabricated in the shops of the American Bridge Company, at Ambridge, Pa. The tests were made on the 2 160 000-lb. hydraulic testing machine of The Phœnix Iron Company, at Phœnixville, Pa.

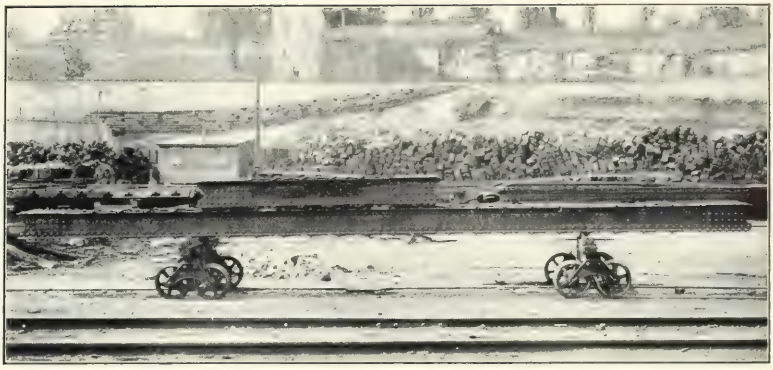


FIG. 1.—LONG-COLUMN TESTS.

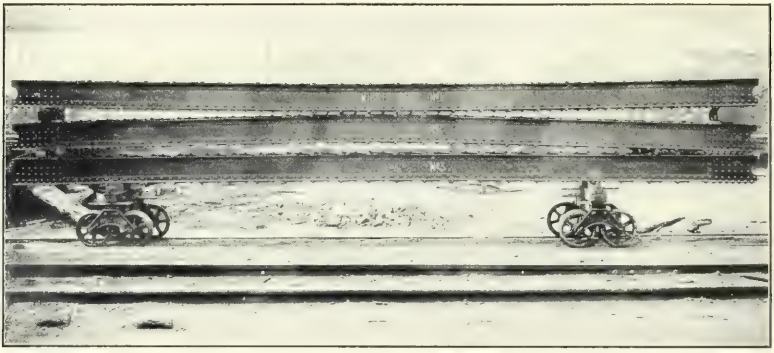


FIG. 2.—LONG-COLUMN TESTS.

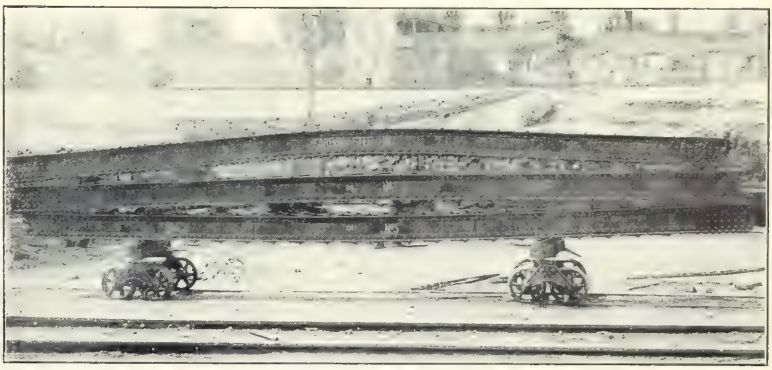


FIG. 3.—LONG-COLUMN TESTS.

It was intended that all material of each kind, except the rivet steel, should be from the same melt. This was not effected; but the record in Table 26 shows that the variations are not greater than would occur in any bridge member.

TABLE 26.—Material Used in the Fabrication of Struts.

	Nickel steel. Heat No.		Carbon steel, Heat No.	
Side plates, 12 by $\frac{3}{8}$ in.	17 673		41 520	
Angles, 3 by 3 by $\frac{3}{8}$ in.	17 749 (1–10 ft.	17 065)	"	
Batten plates, 16 by $\frac{3}{8}$ in	2 116	(23)	"	(1) 45 254
Pin plates, 12 by $\frac{3}{4}$ in	17 673		"	
" " $11\frac{1}{4}$ by $\frac{3}{8}$ in.	17 749		"	
" " 6 by $\frac{3}{8}$ in.	17 673		"	
Lace bars, $2\frac{1}{2}$ by $\frac{3}{8}$ in.	17 749	(55)	"	(6) 60 161
Rivets, $\frac{7}{8}$ in	2 192		Stock.	

Composition.

Heat No.	Nickel.	Carbon.	Manganese.	Sulphur.	Phosphorus.	Silicon.
17 673	3.68	0.41	0.76	0.020	0.005	0.046
17 749	3.52	0.36	0.76	0.030	0.012	0.058
2 116	3.50	0.38	0.70	0.04	0.03	0.04
17 065	4.25	0.463	0.67	0.014	0.019	.....
2 192	3.28	0.20	0.62	0.023	0.010	.....
41 520	none	0.23	0.56	0.025	0.025	0.020
45 254	none	.....	.....	.....	0.028	.....
60 161	none	0.19	0.45	0.030	0.005	.....

Physical Properties.

Heat No.	Elastic limit.	Ultimate strength.	Elongation in 8 in.	Reduction of area.
17 673	67 500	112 600	16.5	50.0
17 749	.....	.....	.....	.....
2 116	58 900	92 800	19.5	28.4
17 065	68 500	108 600	18.75	41.3
2 192	56 300	77 600	25.50	51.3
41 520	44 200	66 200	29.25	56.2
45 254	37 100	61 300	31.75	56.2
60 161	40 700	60 800	30.25	60.8

The bending tests on this material were very satisfactory. Those for the nickel steel are fully detailed elsewhere in this paper. Those for the carbon steel, Heat No. 41 520, bent 180° with an opening of $\frac{1}{4}$ in. before cracks showed. Heat No. 45 254 bent 180° flat without cracking.

The material was received at the shop during January and February, 1905. Shopwork was proceeded with during the latter part of April.

"De Pontibus" specifications for medium-carbon steel were to govern the fabrication, the detail drawing, however, was not so marked, so the punching was started full size. The 6-in. pin-plates for all the struts, and twelve holes in one 10-ft. nickel-steel angle and three holes in another, were punched $\frac{15}{16}$ in. in diameter instead of $\frac{13}{16}$ in. The shop also mis-sheared the nickel-steel pin-plates. These errors, indirectly, caused a delay of one year; and, before the rejected material could be replaced, all work was ordered stopped. The shop was ready to resume in March, 1906, but, during the interval, the rivet rounds, one 10-ft. nickel-steel angle, and several lace-bars were lost. In order to proceed, a 3 by 3 by $\frac{7}{8}$-in. angle from Heat No. 17 065 was used, and two carbon-steel lace-bars were substituted for each missing nickel-steel bar. These were placed at the ends of the struts so as to have a minimum effect on the results. The 6-in. plates of nickel steel were replaced, and the other mis-punched material was used. Additional nickel-steel rivet material was ordered. Carbon-steel rivets were taken from stock. These matters being finally adjusted, shopwork was again taken up in April, and pushed through without mishap. Extreme precautions were taken at every step to keep the two steels separate.

One punch and one die were broken during the punching of the nickel steel. All reaming was done dry, with only graphite lubricant, and with high-speed blue-chip reamers. One reamer could ream to $\frac{15}{16}$-in. about 60 in. of $\frac{13}{16}$-in. holes in the nickel steel before needing regrinding; while, in the carbon steel, it could do four or five times as much. The riveting was done with a machine capable of exerting a pressure of about 35 tons per sq. in.

This riveting work was all done at one time, on the same machines, the rivets being heated in the same fire. No variation was made from regular shop practice. All rivets were tested, but none cut out. The heads of many of the nickel-steel rivets cracked at the edge in making and in driving, but, on the whole, the appearance was very good.

The struts were shipped to Phœnixville on April 21st, where they were tested early in June.

A special cast-steel bolster was made by the Phœnix Iron Company for holding the ends of the struts in position in the machine. The struts, therefore, were free to move vertically, and, in this direction, were the same as round ended, this being the usual condition in pin-connected bridge construction. The short struts were not supported, except at the ends; but the long struts, in order to counteract the bending from their own weight, were counterweighed at two intermediate points, 8 ft. apart. To the eye, these long columns, when in the machine, appeared straight, and a stretched string showed a camber upward not exceeding in any instance more than 0.34 in., and two struts that showed such a camber failed by bending in the opposite direction. It would seem, therefore, that this initial flexure had no influence on the result.

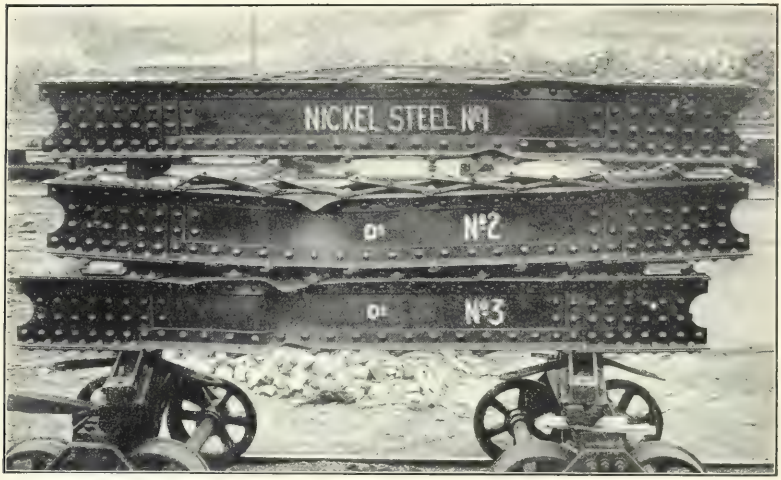


FIG. 1.—SHORT-COLUMN TESTS.

FIG. 2.—SHORT-COLUMN TESTS.

As the load was applied to these long columns, flexure took place, the normal condition being resumed on the removal of the load until a permanent set was evident.

The pressure was registered by a Shaw mercury gauge, graduated to 10 lb., equivalent to 32 000 lb. load on the strut. Intermediate values could be easily read. The accuracy of this gauge was examined and certified to by the maker.

The amount of compression in the struts under each increment of load was measured by a contrivance, designed by Tinius Olsen and Company, consisting of a long arm (7.54 ft. for short struts and 27 ft. for long struts), one end resting immovably on one batten-plate, and the other carrying a pointer resting in a punch mark made in the batten-plate at the other end of the strut and turning on an axis, placed so that any variation in the length of the strut was multiplied five times and measured on an arc divided to hundredths of an inch. This becomes a very delicate apparatus, when the length between bearing points is considered. The movement of the head of the machine varied from $\frac{1}{16}$ to $\frac{3}{8}$ in. per min.

Strut No. 1, the first tested, was the short one of nickel steel, having the $\frac{7}{16}$-in. angle substituted from a heat containing $4\frac{1}{4}\%$ of nickel. The load was applied continuously, and the pointer of the gauge and the mercury column were watched carefully. This procedure was adopted for the first test, with some hopes that the elastic limit could be detected by the movement of the pointer or by the mercury column, or perhaps by both. There was no perceptible change in the rate of movement of either.

The two mis-punched angles were also used in this strut, also carbon-steel lace-bars to replace the lost ones of nickel steel, as described before.

In succeeding tests, loads increasing in amounts were alternately applied and removed, and the movement of pointer noted. This method was very successful in establishing what loads produced a permanent set, also the amount of temporary shortening under these loads. This quantity is the sum of two changes, one resulting from a compression of the material and the other from the bending of the strut. It was thought that a succession of applications of loads in this way would weaken the strut below what it might be expected to carry, so that, in the beginning of the work, a greater difference between the successive loads was adopted. The number was gradually increased, as the maximum strength of the strut did not seem to be affected thereby.

In Table 27 the smallest appreciable permanent set was assumed as 0.005 in. and as a "set" of 0.005 in. in 7.54 ft. corresponds to a set of 0.018 in. in 27.0 ft., the load producing approximately this amount, 0.015 in., is included for comparison.

It will be noticed that in one instance only did a long strut of nickel steel have a set of this amount.

TABLE 27.—Summary of Results of Compression Tests of Struts.

Strut No.	Nickel Steel. Load per square inch producing:			Carbon Steel. Load per square inch producing			Ratio: Nickel steel / Carbon steel	
	Set = 0.005 in.	Set = 0.015 in.	Failure.	Set = 0.005 in.	Set = 0.015 in.	Failure.	Set = 0.005 in.	Failure.
10-Foot Struts.								
1	53 700	.....	68 500	29 600	.....	38 900	.....	.....
2	.....	.....	68 500	29 600	.....	39 800	.....	.....
3	51 800	.....	69 200	27 200	.....	38 900	.....	.....
Average.	52 800	.....	68 700	28 800	.....	39 200	183	175
30-Foot Struts.								
1	39 800	41 600	44 400	.....	.....	29 600	.....	.....
2	42 500	.....	47 200	16 600	22 200	32 400	.....	.....
3	.....	.....	42 500	17 000	20 400	29 600	.....	.....
Average.	41 200	.....	44 700	16 800	21 300	30 500	245	147

The reading of the pointer was discontinued after a marked permanent set was shown, or when the temporary shortening was so much as to exceed the limits of the contrivance; in three instances it was possible to watch the pointer up to the failure of the strut.

The permanent set in two tests was not obtainable, one in the short strut of nickel steel, as already explained, and the other in the first long strut tested—Strut No. 1 of carbon steel. The movement of this strut disturbed the extensometer so that the readings taken are in doubt.

The short struts failed by the buckling of the angles and side plates, in four cases at the middle panel, and in two cases at the panel next to the middle. The first carbon-steel strut crumpled upward, the others sideways.

The long struts failed by bending. The distortion of the pin-hole was not appreciable.

The amount of deflection at the middle of the long strut after failure varied from 0.64 to 2.60 in.

These tests were conducted with great care, and the figures given are, it is believed, correct. They are based on data furnished by the Phœnix Iron Company as to the relation existing between the mercury column and the pressure on the piston of the machine.

An examination of Table 27 shows that there is no evidence of an elastic limit. The total permanent set was exceedingly small, especially in the nickel-steel struts. The readings, however, are very uniform.

The compression in the 10-ft. struts, under equal loads, seems to be about the same for both steels, up to the point where the carbon steel shows a permanent set; and it then becomes greater for the carbon steel. The amount of bending of the long struts is the same for both steels up to the point of failure of the carbon steel.

The results are shown in detail in Tables 51 and 52 and in the photographs, Figs. 1 and 3, Plate CV. The nickel steel in this test compares very favorably, indeed, with the carbon steel, the short struts being three-quarters and the long struts one-half as strong again as those of carbon steel.

These tests were made in June, 1906.

Coefficient of Elasticity.

The coefficient or modulus of elasticity is the ratio of the unit-stress to the unit-deformation, within the elastic limit of the material. As its value varies inversely with the deformation, it may be regarded as a measure of the stiffness of the material. The less the change in length under a given stress the greater is the coefficient of elasticity and the stiffer is the material.*

Six specimens were prepared, three of nickel steel and three of carbon steel, as follows:

No. 158 from the 12 by 1-in. plate of nickel steel, Heat No. 17 673.

B 1 from a 6 by 1-in. unannealed eye-bar of nickel steel, Heat No. 17 749.

B 1 from a 6 by 1-in. annealed eye-bar of nickel steel, Heat No. 17 749.

No. C 196 from a 12 by 1-in. plate of carbon steel, Heat No. 33 342.

No. C 162 from a 12 by 1-in. plate of carbon steel, Heat No. 33 342.

No. C 109 from a 12 by ¾-in. plate of carbon steel, Heat No. 33 342.

The specimens were turned for a distance of 9½ in. between fillets to a diameter as large as possible. This form was used because the construction of the Olsen extensometer made a round specimen desirable, and because the area of a turned round may be determined with greater accuracy than that of a rectangular section.

The loading was applied very slowly, and all readings were taken with the beam balancing. Within the elastic limit, a given load may be sustained for an indefinite period, but, as the piece begins to yield, the stretching relieves a part of the load. Restoring it again increases

*Merriman.

the stretch, and this condition continues until an equilibrium is established, when the beam will remain "balancing."

The extensometer read directly to $\frac{1}{1000}$ in. and by means of a vernier to $\frac{1}{10000}$ in. The first reading was always taken with a given load on the specimen—4 000 lb. for the larger pieces, and 2 000 lb. for the carbon-steel pieces. In one of the tables filed for reference in the Library of the Society, however, all readings have been reduced to an assumed zero stretch with no load on the specimen. The values obtained are not scientifically exact. They are as nearly correct as the apparatus and careful observations could make them. In several instances two sets of values for the coefficient are recorded; the *"a"* results are more nearly in agreement and it is believed are the more accurate. With pieces, *"B* 1," the range over which the *"b"* readings were taken is small, and, though a change seems to take place at 12 000 lb., it is only apparent, and, had intermediate readings been taken, it is probable that a series would have been obtained similar to those between 12 000 and 25 000 lb. In the case of the carbon steel, after the *"b"* readings had been taken, the load was removed and a second series taken as a check on the first, because of suspected errors—through slipping of either the apparatus or the grips. The range for the carbon steel was necessarily small, and it is believed that a more delicate apparatus should be used for such material.

The values for the two steels are:

For the nickel steel..	$E =$	30 075 000,	1-in. plate, No. 158.
	$=$	30 440 000,	6 " E. B., B 1, Original.
	$=$	30 420 000,	6 " E. B., B 1, Annealed.
For the carbon steel..	$E =$	28 940 000,	1 " plate, C 196.
	$=$	29 480 000,	1 " plate, C 162.
	$=$	29 840 000,	$\frac{3}{4}$ " plate, C 109.

The coefficient for the nickel steel is slightly higher, but the difference is not great; and, as probable errors in measurement of specimens or loads may be as high as 500 000, it may be assumed that below the elastic limit the two steels behave almost identically. Beyond the elastic limit, however, the yielding of the nickel steel is more gradual, and is smaller in amount.

Had the entire range, up to the yield point, been taken in each case, the nickel steel would still be found to have the higher coefficient, the difference being greater than before.

It will be noticed that the elastic limit, as determined by this method, is considerably below the yield point. Curves plotted from these readings show clearly the relationship between the two. After the amount of stretch up to the yield point had been noted, the tests were continued and the specimens broken with the results shown in Table 28:

TABLE 28.—Results of Tests for Coefficient of Elasticity.

Mark.	Area, in square inches.	Load per Square Inch.			Elastic Ratio.		Elongation, in 8 in., per cent.	Reduction of area, per cent.	Fracture.
		Elastic limit, in pounds.	Yield point, in pounds.	Ultimate strength, in pounds.	Elastic limit, per cent.	Yield point, per cent.			
Nickel Steel.									
158....	0.760	36 800	56 600	104 600	35.2	54.1	17.3	47.9	S Cup.
B 1....	0.722	40 200	62 300	104 300	38.5	59.7	18.8	53.3	S ½ Cup Unannealed.
B 1....	0.727	35 800	56 400	102 800	34.8	54.9	18.8	47.0	S ½ Cup Annealed.
Carbon Steel.									
C 196..	0.754	15 900	22 500	58 100	27.4	38.7	30.8	58.6	S ½ Cup.
C 162..	0.590	11 900	25 400	59 000	20.2	43.1	32.5	59.7	S ¾ Cup.
C 109..	0.383	15 700	28 700	65 300	24.0	44.0	28.0	60.3	S Cup.

Comparing the results in Table 28 with those obtained in the tensile tests of A. A. S. M. specimens, it is seen that there is a close agreement in the case of the nickel steel, Table 56; in the case of the carbon steel, the yield point, as determined above, is lower than that obtained from the A. A. S. M. specimens, and this could have been anticipated, as the speed of the machine was different. The method just described allowed the full deformation of the piece to take place before the given load was increased; while, in the previous tests, the increase in load was continuous.

The tests of turned rounds also gave lower values for the yield point and ultimate strength, because of the removal of the rolled surface or skin.

From these tests it is also seen how, under certain conditions, the ratio of the elastic limit or the yield point to the ultimate strength becomes, for the nickel steel, 50% higher than for the carbon steel.

The very low elastic ratio reported in Table 31 for the 1-in. plates of carbon steel may be partly accounted for by the speed of the machine, having been 1 in. in 20 min., for these pieces, and 1 in. in 6 min. for the other pieces. The tests made at the plant of the Luken's Iron and Steel Company, Table 33, show, however, that not all the drop may be accounted for in this way.

It would be interesting to experiment further in this direction, and to obtain the amount of permanent set under given increments of load. Only two attempts were made to obtain data of this kind.

The load was removed from piece No. 158 after 30 000 lb. (39 500 lb. per sq. in.) had been reached and a set of 0.0001 in. was noted. The removal of load from *B* 1 (original bar) after 40 000 lb. (55 400 lb. per sq. in.) gave a set of 0.0018 in.

Further attempts were abandoned because of the danger of serious derangement of the extensometer and the interruption to the continuity of the readings.

Tests of Eye-Bar Material.

This material was rolled by the Carnegie Steel Company from the two 51-ton heats, Nos. 17 673 and 17 749, known as "ideal shape nickel steel."

The following sizes were tested:

No.	Size.	Heat No.	Size of ingot.	Weight of ingot.	Size of slab.	Weight of slab.
1	16 by 2 in.	17 673	25 by 30 in.	12 080 lb.	$18\frac{1}{4}$ by $14\frac{1}{4}$ in.	3 580 lb.
3	8 by 2 in.	17 749	22 by 25 in.	8 200 lb.	10 by 8 in.	1 900 lb.
4	6 by 1 in.	17 749	18 by 20 in.	6 060 lb.	10 by 8 in.	1 345 lb.

The bars were forged and tested at the Ambridge Plant of the American Bridge Company.

Forging of 6-in. Eye-Bars.—On June 14th, 1906, the 6-in. bars were taken into the shop. The eighteen months' exposure had rusted the surface considerably.

The general method followed in forging these bars differed only from the usual practice in that the desired temperature was obtained more slowly. Along with the four nickel-steel bars, sixteen plain carbon-steel bars were heated and forged.

Each bar was stamped with a distinguishing mark—*B* 1, *B* 2, *B* 3 and *B* 4—for identification. From one end of each, two pieces, 18 in. long, were sheared for specimen tests, one piece being tested as rolled and the other after having been annealed with the forged bars.

The furnace used for heating was fired with gas. The bars were run in at 11.08 A. M. in couples, *B* 2 on *B* 3 and *B* 4 on *B* 1, the carbon-steel bars being put in at the same time.

At intervals, the temperatures of furnace and bars were obtained by the Le Chatelier electrical pyrometer, and, when this failed, by a Queen and Company optical pyrometer.

First Heat.—The temperature of the furnace at 10.30 A. M. was 1 200° (650° cent.), at 11.20, twelve minutes after the bars had been entered, it was 1 115° (600° cent.). The temperature at the time of forging was not obtained, as, through a misunderstanding, the heater removed the bars before the stated time. After the rough heads (known as cobbles) had been formed on the pile of two bars, they were then rolled, still together, and then brought back to the furnace. Practically all the upsetting was done at this operation, the second upsetting simply perfecting the shape of head.

The second heating was carefully watched; the "cobbles" were entered about 12.15 P. M.; at 12.40 P. M. the temperature of the furnace was 1 960° (1 070° cent.); at 12.52 P. M., it was 1 975° (1 080° cent.);

and after this reading the thermo-junction of the pyrometer separated. At 1.20 P. M. the temperature was 2 000° (1 090° cent.); at 1.30 P. M. 2 100° (1 150° cent.); and at 1.35 P. M., 2 100° (1 150° cent.). Bars *B* 4 and *B* 2 were then removed for final upsetting. The furnace was then forced to a higher temperature, reaching 2 200° (1 200° cent.) at 1.42 P. M., when Bar *B* 1 was removed. At 1.45 P. M., Bar *B* 3 was removed, the temperature being about 2 250° (1 230° cent.). The head of Bar *B* 1 was dished in punching, which necessitated reheating and flattening under a hydraulic ram.

After the final upsetting, the bars were rolled singly and then punched, and the ears were sheared off, after which each bar was again rolled and laid on edge on skids until the entire heat was forged.

The scale formed during the heating was removed by rapid hammering both before and after upsetting.

Annealing of 6-in. Eye-Bars.—In order, if possible, to obtain a variation in the heat treatment of the bars, they were run into the furnace in pairs, with the short lengths for specimen tests on top, as shown in Fig. 74.

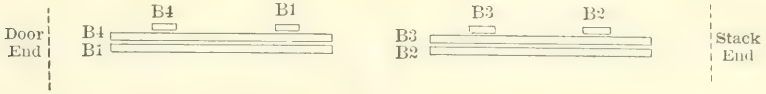


FIG. 74.

Bars *B* 1 and *B* 2 rested on the rolls and were separated from Bars *B* 4 and *B* 3 by ½-in. square rods. The specimen pieces were placed about 4 ft. from the ends of the bars.

The free circulation of the gases through the pin-holes and the possible overheating of the head were prevented by placing caps, 4 in. square, over the holes.

All temperature readings were taken with the electrical pyrometer; this was calibrated before being used. The gun of the pyrometer was inserted through small rectangular openings, about 4½ in. square, 6 ft. apart and 15 in. above the tops of the rolls and 13 in. above the eye-bars; the temperature of the furnace recorded, therefore, is that about 1 ft. above the top bar. The place occupied by the bars was probably from 50 to 100° (10 to 40° cent.) cooler than that indicated.

The bars were rolled into the furnace at 9.40 A. M., June 16th. The furnace had been operated continuously during the night, and was, therefore, thoroughly heated.

The heating of Bars *B* 4 and *B* 1 occupied 3 hr. 23 min., and of Bars *B* 3 and *B* 2, 3 hr. 50 min.

The probable average temperatures of the bars, when withdrawn, were:

B 1.....	1 500°	(820° cent.)
B 2.....	1 450°	(790° ")
B 3.....	1 500°	(820° ")
B 4.....	1 550°	(840° ")

Forging of 8 by 2-in. Eye-Bars.—The same marks that were stamped on the 6-in. bars were put on the 8 by 2-in. bars for identification. These bars were heated in an oil-fired furnace somewhat in need of repairs, so that the heating could not be controlled accurately; the extreme ends were much the hotter. The bars were heated and forged by themselves.

In the first head:

Bar *B* 1 was in the furnace 74 min.,
" *B* 2 " " " " 76 "
" *B* 3 " " " " 83 "

The average length of each interval required for upsetting, rolling, etc., was 6 min., or a total of 18 min. for each bar.

Reheating was made necessary because of the great width of head relative to the width of bars, the excess being about 80 per cent.

On account of the poor condition of the furnace, the bars were shoved in about 3 in. further than usual.

In the second head:

Bar *B* 1 was in the furnace 58 min.,
" *B* 2 " " " " 59 "
" B 3 " " " " 77 "

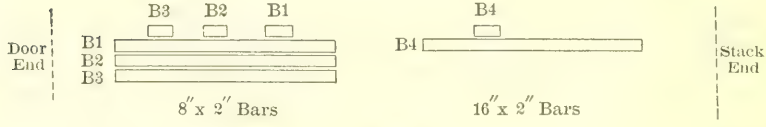


FIG. 75.

The average length of each interval required for upsetting, rolling, etc., of Bars *B* 1 and *B* 2 was 4 min., or a total of 12 min. for each bar. Bar *B* 3 was heated a fourth time, and the total time for upsetting, etc., was about 21 min.

The total time occupied with both heads was 3 hr. 27 min.

Forging of 16 by 2-in. Eye-Bar.—The 16 by 2-in. bar was heated and forged with five plain carbon-steel bars. The heating was done in the furnace used for the 8 by 2-in. bars, and the upsetting, etc., were done by the same men and machines. For identification it was stamped *B* 4.

The first head was in the furnace 1 hr. 45 min., the second, 91 min. The forging of the first head occupied 22 min., in three intervals; the second occupied 30 min., in five intervals.

The total time occupied was 4 hr. 10 min.

Annealing of 8 by 2-in. and 16 by 2-in. Bars.—These bars were annealed in the furnace already described. The heat treatment differed slightly, a higher temperature being attained.

At 10.25 P. M. the bars were rolled into the furnace in the order shown in Fig. 75.

The length of time the bars were in the furnace was 5 hr. 25 min.

At the time of withdrawal the temperatures of the heads of the bars nearest the door were, for Bar *B* 1, 1 500° (820° cent.); *B* 2, 1 440° (780° cent.), and *B* 3, 1 400° (760° cent.). The temperature of the 16-in. bar was not taken because of the breaking of the thermo-junction. Its temperature, however, was probably 1 600° (870° cent.), the stack end of the furnace being approximately 100° (40° cent.) higher than where the other bars were. The approximate average temperatures of the bars were probably as follows: *B* 1, 1 550° (840° cent.); *B* 2, 1 500° (810° cent.); *B* 3, 1 450° (790° cent.); *B* 4, 1 650° (900° cent.).

Tensile Tests of Full-Sized Eye-Bars.—These bars were tested in the new hydraulic testing machine of the American Bridge Company, at Ambridge, Pa. The pressure was supplied by a short-stroke, two-cylinder, Deeming hydraulic pump geared to an electric motor. The capacity of the machine is 4 000 000 lb., the pressure being read on a Shaw mercury gauge.

The only nickel-steel bars manufactured (within the writer's knowledge) up to the present time are those made by the American Bridge Company for the Blackwell's Island Bridge, New York City. With a knowledge of the difficulty in obtaining the yield point of nickel steel from the movement of the mercury column, it was agreed between the manufacturer and the engineer that an extensometer should be used for establishing this point, and that it should be assumed to be the load producing a permanent set of 0.025 in. in 20 ft. This method has not given entire satisfaction, because of the unusually low elastic limits reported at times.

It was intended, in this experimental work, to follow the same procedure; but the results thus obtained when considered in their relation to the readings as a whole and to the physical condition known as peeling are inconsistent. It is believed that in several instances the yield point had not been reached when the extensometer had recorded a set of 0.025 in. in 20 ft.

At the yield point there is always a marked change in the amount of stretch when the load is increased uniformly, and, between the elastic limit and this point, the stretch becomes gradually greater for each increment of load. Just before and at the yield point, the mill or furnace scale begins to fall from the bar because of its unelasticity. This is a certain indication that the bar is stretching rapidly.

In Tables 53 and 56, therefore, the results have been worked out on this basis, and not on an arbitrary assumption based on a certain permanent set.

This method for determining the elastic limit or yield point was the same as that used in the tests of struts, the load in increasing amounts being alternately applied and removed, and the stretch and

permanent set at each application noted by the extensometer. The readings are given in Tables 54 and 55, corrected, however, so that the zero readings for stretch and permanent set occur under no load.

Actually, the pointer was set in advance of the zero mark, as it was found impossible to set the extensometer pointer exactly at this mark. It was also found, in several instances, that, after a small load had been put on the bar, the pointer returned so far as to pass the original setting, and in the case of the 6-in. bar, *B* 3, it passed the limit of the graduated portion so that all the readings had to be estimated by the eye.

The several readings taken at two points within the elastic limit of the 8-in. and 16-in. bars, *B* 1 to *B* 4, Table 55, furnish some information as to the probable error of these results. It will be seen that differences of several thousandths occur. Such a series of readings for permanent set as occur for the 6-in. bar, *B* 1, where, between loads of 300 000 and 330 000 lb., there was no change, furnish another reason why too much importance should not be given to these results by themselves.

The complete graduated arc measured an extension of but 0.5 in., hence the readings were limited to this amount, which is far too small, it is thought, for obtaining all the information necessary in determining accurately the yield point. The readings for each bar were discontinued only when the limit of the extensometer had been reached.

Another objection to this procedure was the inability of the operator to sustain a constant pressure sufficiently long for a yielding of the material to take place, as described under "Coefficient of Elasticity" determinations. This was especially the case with the small bars; it was impossible to reduce the speed of the machine to what it should have been for experimental work. The speed could be varied within certain limits by an electric controller in connection with the motor operating the pump; and it also varied with each increase of load. Just before the maximum load for the 6-in. bars was reached the movement of head was about 3 in. in 1 min.; for the 8-in. bars, $1\frac{1}{4}$ in. in 1 min.; and for the 16-in. bars about $\frac{1}{2}$ in. in 1 min. The 6 and 8-in. bars broke with a uniform silky fracture, and an average amount of elongation and reduction of area; and the drop of ultimate strength and elastic limit below those of the specimen unannealed tests was fairly uniform, so that the annealing described was evidently properly done. The number of tests is too small to draw any conclusions as to the effects of forging and annealing temperatures, the differences between the individual bars being very small.

The fine crystalline fracture of the 16-in. bar would give reason for thinking that it should have been held for a longer time in the annealing furnace and at a lower temperature. The fracture was

uniform, and gave no indication of maltreatment; but the elongations are greater than those of the smaller bars.

Coefficient of Elasticity—Full-Sized Bars.—It was desired to obtain the coefficient of elasticity in tension from these tests. For this purpose ten readings with the extensometer were taken of the stretch in each of the 8 and 16-in. bars, five with a given load as small as practicable, and five with a load just below the elastic limit (Tables 54 and 55). The readings vary slightly, as already noted, but, after excluding the doubtful ones, an average was used for the coefficient determination.

The following coefficients of elasticity were obtained:

8 by 2-in.,	$B\,1 - E =$	26 900 000
" " " "	$B\,2 - E =$	27 910 000
" " " "	$B\,3 - E =$	27 790 000
16 " 2 "	$B\,4 - E =$	26 770 000

and, similarly, from the 6 by 1-in. bars, the approximate values:

6 by 1-in.,	$B\,1$ and $B\,4 - E =$	27 500 000
6 " " "	$B\,2$ and $B\,3 - E =$	25 800 000

These values are much below the 30 000 000 obtained from the 1-in. round specimens cut from the 6-in. bar, $B\,1$.

The most reasonable explanation of this difference is that the areas used in the calculations are too large. These were taken with calipers after all loose scale had been knocked off, and were the same as those used in obtaining the physical properties. The actual area of cross-section is always less than the section measured, because of the heavy scale resulting from rolling and annealing. Assuming the coefficient of elasticity to be 30 000 000, as obtained from the specimens, the extensometer readings would indicate that the loads per square inch given in Tables 53 and 56 are about 9% too small for the 6-in. bars and 7% and 6% too small for the 8-in. and 16-in. bars. Some of the errors may arise from the extensometer readings, but the fact that there is a close agreement among the bars of the same size would point to the first explanation. This scale is difficult to remove. It may be as much as $\frac{1}{8}$ in. thick on the heads; and a scale 0.04 in. thick would be sufficient to account for this difference.

Tensile Tests of Specimens of Eye-Bar Material.—The tensile tests of the specimens cut from the 6-in. bars, $B\,2$, $B\,3$ and $B\,4$, were made by The Riehle Brothers Testing Machine Company of Philadelphia. The results are disappointing because of the failure of the Riehle autographic apparatus to work properly. The yield point had to be estimated from measurements obtained by a pair of dividers. The specimens from 8-in. and 16-in. bars, with the exception of $EB\,32$, were tested at Drexel Institute on a 200 000-lb. Olsen machine

with an autographic apparatus. The two curves obtained for Specimens *EB* 11 and *EB* 21 are very different from the others at the yield point and, therefore, are marked abnormal. The variation in speed was not sufficient to affect the results materially.

The yield point, ultimate strength, and elastic ratio, Table 57, are lower for the annealed specimens in every instance but one (*EB* 32 broken by The Riehle Brothers Company). The elastic ratio of this piece is higher; but the yield point is in doubt, as no dividers were used. The elongation in 8 in. of the annealed specimens is less than in the unannealed in all but four instances. In two of these it is equal, and in the other two but 0.2% and 3.3% higher; and in the latter case, the fracture of the unannealed bar occurred near the end gauge mark, so that the stretch on one side was limited by the fillets. The reduction of area of annealed bars is lower in all but two instances—the two pieces cut from the 8-in. bar, *B* 3.

The differences are very small, and the large elongations and reductions of both the annealed and unannealed bars are indicative of a good quality of material.

The location of these specimen pieces in the eye-bar is a matter of interest. The center line of specimens cut from the 6-in. bars was located 1½ in. from the edge; from the 8-in. bars, two pieces were cut, one from the edge and one from the middle of the bar; and from the 16-in. bar only one 18-in. length was cut, and this was again sheared along the middle line, giving two pieces, 8 by 2 by 18 in., one of which was annealed with the bar. Pieces for bending tests were cut from the rolled edge, and the tensile pieces marked "2" next to these and 3 in. from the rolled edge. The piece marked "1" was cut from the sheared edge, so that *EB* 41 and *AEB* 41 were adjacent in the bars; the former being tested as rolled and the latter being annealed.

The yield point and tensile strength of the pieces cut from the edge and middle of the same bar are different. Edge pieces, with the exception of pieces *EB* 22 and *AEB* 22, gave lower results. These differences are practically eliminated by annealing.

TABLE 29.—Comparison of Results of Tests of Specimens.

Physical property.	Specimens Cut from 8 in. and 16 in. Eye-Bars.				Specimens Cut from 6-in. Eye-Bars.	
	Middle.		Edge.			
	Original.	Annealed.	Original.	Annealed.	Original.	Annealed.
Yield point.....	60 100	53 600	55 600	52 900	60 600	54 800
Ultimate strength..	101 500	97 800	98 500	96 600	103 100	100 000
Elastic ratio.....	58.7%	54.9%	56.4%	54.8%	58.7%	54.5%
Elongation in 8-in...	21.1%	20.1%	21.9%	21.0%	19.4%	19.1%
Reduction of area..	48.5%	47.5%	49.7%	47.4%	51.1%	48.0%

Bending Tests of Eye-Bar Material.—As already described, pieces 18 in. long were cut from each bar before forging. One piece was annealed with the bar from which it was cut, but the other received no heat treatment whatever.

The specimens for bending were taken from the edge, and one side only was machined. They were 2 by 18 in. by the thickness of the bar.

The bending was done in the hydraulic bending machine at Pencoyd, Pa., in the American Bridge Company's shop.

A mandrel with end rounded to a radius of 2 in. was used for the specimens 2 in. thick, and the anvil was U-shaped with a mouth $8\frac{1}{2}$ in. wide. The casting was broken in the first attempt. A similar, but heavier, anvil with a $9\frac{1}{2}$-in. mouth was used throughout the series.

Each specimen was bent as much as possible by being forced into this opening by the 4-in. plunger, and then bent farther by pressure on the ends. It was desired that the radius of bend should be reduced to the thickness of the material; and in every instance the bending was greater than this.

The specimens from the 6-in. bars were similarly bent, the plunger being 2 in. instead of 4 in. in diameter, and the aperture of anvil being reduced to $5\frac{3}{4}$ in., thus increasing the amount of bending about the end of plunger. No definite amount of bending was contemplated, therefore some of the pieces were bent to cracking. Table 58 and the photograph of the bent specimens shown by Fig. 2, Plate CIV, give in detail the results of this bending.

The radius of the inner surface of the bends varied from 0.4 to 0.7 of the thickness of specimen, thus satisfying the usual requirements demanded of a medium carbon steel of 60 000-lb. ultimate strength.

APPENDIX A.

TABLE 30.—Tensile Tests on A. A. S. M.

Made on 200 000-lb. Olsen

Test.	Original.		Load per Square Inch.		Elastic Ratio.	Percentage of Elongation in:				Reduction of Area.
	Section, in inches.	Area, in square inches.	Yield point, in pounds.	Ultimate strength, in pounds.	Per cent.	2 in.	4 in.	6 in.	8 in.	Per cent.
12-in. Universal Plates.—Heat No. 17 673.										
Minimum	1.510 × 0.385	0.581	.....	115 200	.....	25.0	19.5	16.7	15.3	45.1
Average of 4.	1.510 × 0.385	0.581	.....	116 300	.....	29.3	20.8	18.0	15.8	46.4
Maximum	1.510 × 0.385	0.581	.....	117 200	.....	32.5	22.5	19.3	16.8	47.5
Minimum	1.505 × 0.385	0.579	66 700	112 800	58.8	21.0	19.5	16.7	13.8	43.9
Average of 4.	1.505 × 0.385	0.579	67 875	114 550	59.2	26.0	20.7	17.2	14.8	45.2
Maximum	1.505 × 0.385	0.579	70 100	116 600	60.1	29.0	21.5	17.7	15.6	45.9
Minimum	1.510 × 0.505	0.763	64 200	110 600	58.0	30.0	20.5	17.7	15.5	45.2
Average of 4.	1.510 × 0.505	0.763	64 200	113 020	58.0	31.0	22.7	18.5	16.4	46.2
Maximum	1.510 × 0.505	0.763	64 200	114 700	58.0	33.0	24.0	20.3	18.0	46.9
Minimum	1.500 × 0.505	0.758	61 400	106 700	57.3	26.0	23.5	19.0	16.6	49.1
Average of 3.	1.500 × 0.505	0.758	62 060	107 830	57.5	28.5	23.7	19.2	16.7	49.8
Maximum	1.500 × 0.505	0.758	62 700	108 400	57.8	30.0	24.0	19.5	16.8	50.7
Minimum	1.510 × 0.750	1.133	61 400	106 000	57.2	31.0	23.5	18.7	16.3	42.5
Average of 4.	1.510 × 0.750	1.133	61 830	106 900	57.9	33.5	25.0	20.3	17.5	46.6
Maximum	1.510 × 0.750	1.133	62 000	107 700	58.5	36.0	27.0	21.7	18.3	49.4
1 test only.	1.500 × 0.750	1.125	62 700	108 400	57.8	27.0	24.3	20.7	18.1	47.4
Minimum	1.250 × 1.005	1.256	55 400	106 800	51.6	37.0	26.0	21.3	19.3	45.8
Average of 4.	1.250 × 1.005	1.256	58 550	107 320	54.5	38.5	27.6	22.7	20.0	47.4
Maximum	1.250 × 1.005	1.256	61 600	107 900	57.4	40.0	30.5	24.7	21.5	48.2
Minimum	1.250 × 1.000	1.250	58 400	104 600	55.8	38.0	28.0	21.7	19.8	48.9
Average of 2.	1.250 × 1.000	1.250	58 500	104 600	55.9	39.0	28.7	22.0	19.8	49.1
Maximum	1.250 × 1.000	1.250	58 600	104 600	56.0	40.0	29.5	22.3	19.8	49.4
6 by 6 by $\frac{3}{4}$-in. Angles.—Heat No. 17 749.										
Minimum	1.510 × 0.750	1.133	60 700	102 000	59.5	30.0	24.0	20.0	18.0	40.8
Average of 4.	1.510 × 0.750	1.133	62 250	102 500	60.6	32.0	24.6	20.8	18.6	42.0
Maximum	1.510 × 0.750	1.133	65 600	103 700	63.3	34.0	25.0	22.0	19.0	42.7
8 by 8 by 1-in. Angles.—Heat No. 17 749.										
Minimum	1.250 × 1.019	1.273	53 300	97 100	54.6	28.0	27.5	22.7	20.0	46.0
Average of 4.	1.250 × 1.019	1.273	54 100	97 870	55.3	34.6	28.7	23.7	21.3	48.5
Maximum	1.250 × 1.019	1.273	55 000	98 800	56.6	41.0	30.5	25.3	22.8	51.1

Note.—This table is condensed from the original by giving, for each group of tests,
* Distance from nearest end gauge mark. † Fracture slightly crystalline.

(PART II.)

Standard Specimens of Structural Steel.

Machine at Drexel Institute.

Fracture. Character.	Location. Inches.*	Speed of breaking.	Location of specimen.	Remarks.
S. ½ Cup.	2.2	1 in. in 6 min. to	Edge.	Yield point lost.
S. ¾ Cup.	2.9	ultimate strength.	"	" " "
S. 1 Cup.	3.3	then 1 in. in 2 min. to end.	"	" " "
S. 1 Cup.†	1.2	1 in. in 6 min. within yield point.	Interior	Yield point well marked.
S. ¾ Cup.	2.2	1 in. in 20 min. before and well	Interior.	
S. Ang.	....	beyond yield point, then 1 in.	Edge.	
S. ¾ Cup.	3.3	in 6 min. to end.	Interior.	
S. ¾ Cup.	3.2	1 in. in 6 min. throughout. 1 in. in	Edge	Yield point fairly well marked.
S. 1 Cup.	4.0	6 min. well beyond yield point,	"	Yield point lost.
S. 1 Cup.	4.4	then 1 in. in 2 min. to end.	"	" " "
S. ¾ Cup.	2.8	1 in. in 6 min. within yield point.	Interior.	Yield point fairly well marked.
S. ⅞ Cup.	3.8	1 in. in 20 min. before and well	"	" " " " "
S. 1 Cup.=	4.3	beyond yield point; 1 in. in 6	"	" " " " " "
		min. again; finally 1 in. in 2		
		min.		
S. ½ Cup.	2.5	1 in. in 6 min, throughout.	Edge.	Yield point lost.
S. ⅞ Cup.	3.5	1 in. in 6 min. within yield point;	"	Yield point well marked.
S. ¾ Cup.	4.4	1 in. in 20 min. before and well	"	" " " "
		after yield point; then 1 in. in		
		6 min. to end.		
S. ½ Cup.	2.5	1 in. in 6 min., 1 in. in 20 min., 1	Edge.	Yield point well marked.
		in. in 6 min.; finally 1 in. in 2		
		min. See No. 52.		
S ½ Cup.	3.5	1 in. in 6 min. within yield point.	Edge.	Yield point well marked.
S. ¾ Cup.	3.8	1 in. in 20 min. before and well	"	" " " "
S. 1 Cup.	4.7	beyond yield point; then 1 in.	"	" " poorly marked.
		in 6 min.; finally 1 in. in 2 min.		
S. ⅛ Cup.	4.1	1 in. in 6 min. within yield point.	Interior.	Yield point well marked.
S. ¾ Cup.	4.3	1 in. in 20 min. before and well	"	" " " "
S. 1 Cup.	4.5	beyond yield point; then 1 in.	"	" " " "
		in 6 min. to end.		
S. = Cup.	1.5	1 in. in 20 min. beyond yield point.	Edge.	Yield point fairly well marked.
S. = Cup.	2.3	1 in. in 6 min. to end.	"	" " " " " "
S. = Cup.	3.5	1 in. in 6 min. within yield point;	"	" " " " " "
		1 in. in 20 min. before and well		
		beyond yield point; then 1 in.		
		in 6 min. to end.		
		1 in. in 6 min. within yield point.		
		1 in. in 20 min. before and well		
		beyond yield point; then 1 in.		
		in 6 min. to end.		
S. ½ Cup.	3.6	1 in. in 6 min. within yield point.	Edge.	Yield point fairly well marked.
S. ¾ Cup.	3.9	1 in. in 20 min. before and well	"	" " " " " "
S. 1 Cup.	4.3	beyond yield point; then 1 in.	"	" " " " " "
		in 6 min.; finally 1 in. in 2 min.		
		to end.		

minimum, average and maximum records, as shown in the first column.

TABLE 31.—Tensile Tests on A. A. S. M.
Made on 200 000-lb. Olsen
All Specimens Lo-

Test.	Original.		Load per Square Inch.			Elastic Ratio.	
	Section, in inches.	Area, in square inches.	Yield point, in pounds.	Drop of beam, in pounds.	Ultimate strength, in pounds.	Yield point, per cent.	Drop of beam, per cent.
12-in. Universal Plates.—Heat No. 33 342.							
Minimum	1.510 × 0.355	0.536	37 300	39 700	62 700	51.9	61.7
Average of 4	1.510 × 0.355	0.536	38 800	40 800	63 720	60.5	64.1
Maximum	1.510 × 0.355	0.536	41 400	42 500	64 400	66.1	67.8
16-in. Universal Plates.—Heat No. 41 520.							
Minimum	1.510 × 0.380	0.574	34 500	35 900	66 200	52.1	54.2
Average of 2	1.510 × 0.380	0.574	35 350	36 250	66 750	52.9	54.3
Maximum	1.510 × 0.380	0.574	36 200	36 600	67 300	53.8	54.4
12-in. Universal Plates.—Heat No. 33 342.							
Minimum	1.510 × 0.475	0.717	36 500	37 100	63 000	57.6	58.1
Average of 4	1.510 × 0.475	0.717	36 880	37 320	63 770	57.8	58.5
Maximum	1.510 × 0.475	0.717	37 000	37 700	64 200	57.9	58.9
Minimum	1.510 × 0.755	1.140	32 500	33 700	61 900	52.5	54.4
Average of 4	1.510 × 0.755	1.140	33 420	34 050	62 150	53.8	54.7
Maximum	1.510 × 0.755	1.140	34 200	34 400	62 500	54.9	55.2
Minimum	1.250 × 1.000	1.250	26 400	27 000	59 200	43.3	45.4
Average of 6	1.250 × 1.000	1.250	27 380	29 230	60 850	44.9	47.9
Maximum	1.250 × 1.000	1.250	28 200	31 500	62 490	46.4	50.5
Minimum	1.250 × 0.995	1.244	25 700	26 900	58 500	43.8	45.5
Average of 2	1.250 × 0.995	1.244	25 700	26 950	58 600	43.8	45.8
Maximum	1.250 × 0.995	1.244	25 700	27 000	58 700	43.9	46.2

Note.—This table is condensed from the original by giving, for each group of tests,
* Distance from nearest end gauge mark.

STANDARD SPECIMENS. STRUCTURAL CARBON STEEL.

Machine at Drexel Institute.

cated at Edge.

Percentage of Elongation in:				Reduction of area, per cent.	Fracture.		Speed of breaking.
2 in.	4 in.	6 in.	8 in.		Character.	Location, inches.*	
43.0	35.0	30.3	26.9	55.0	S. Ang.	4.2	1 in. in 6 min to ultimate strength, then 1 in. in 2 min. to end.
45.7	35.7	31.1	27.9	56.2	S. Ang.	4.6	
47.5	37.5	32.7	29.4	56.9	S. Ang. / S. ½ Cup.	4.9	
48.0	35.0	30.7	27.0	52.4	S. Ang.	3.2	1 in. in 6 min. to ultimate strength, then 1 in. in 2 min. to end.
49.0	35.7	31.0	27.2	55.3	S. Ang.	3.5	
50.0	36.5	31.3	27.5	58.2	S. Ang.	3.9	
50.0	35.0	29.0	26.3	54.2	S. Ang.	1.3	1 in. in 6 min. throughout.
51.2	37.1	30.4	27.3	56.5	S. Ang.	2.7	1 " " 6 " "
53.0	38.5	32.0	28.8	58.4	S. Cup. / S. ½ Cup.	3.8	1 " " 6 " " / 1 " " 6 " "
52.0	41.0	34.3	31.3	57.1	S. Cup.	3.8	1 " " 6 " "
53.7	42.0	35.3	32.2	58.0	S. Cup.	4.3	1 " " 6 " "
57.0	43.0	36.3	33.0	58.7	S. Ang. / S. Cup.	5.1	1 " " 6 " " / 1 " " 6 " "
47.0	41.0	34.0	31.0	56.3	S. ½ Cup.	4.2	1 in. in 6 min. within yield point; 1 in. in 20 min. before and well beyond yield point; 1 in. in 6 min. again; finally 1 in. in 2 min.
54.9	42.5	36.0	32.4	57.7	S. ¾ Cup.	4.7	
60.0	44.0	38.3	34.0	59.9	S. 1 Cup.	5.0	
58.0	42.5	36.7	32.5	60.4	S. Aug.	4.3	1 in. in 6 min. within yield point; 1 in. in 20 min. before and well after yield point; finally 1 in. in 6 min.
60.0	43.0	37.0	33.1	60.9	S. ½ Cup.	4.4	
62.0	43.5	37.3	33.8	61.5	.....	4.5	

minimum, average, and maximum records as shown in the first column.

TABLE 32.—Tensile Tests on "Parallel-Sided" Specimens of Structural Nickel Steel. Made on 200 000-lb. Olsen Machines at Pencoyd Iron Works and at Lukens Iron and Steel Company.

Test.		Original. Section, in inches.	Original. Area, in sq. in.	Load per Square Inch. Drop of beam, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio, per cent.	Elongation in 8 in., per cent.	Reduction of area, per cent.	Fracture. Character.	Fracture. Location, Inches.*	Speed of Breaking. At drop of beam, 1 in. in :	Speed of Breaking. At break, 1 in. in :	Remarks.
12-in. Universal Plates.—Heat No. 17 673.	Minimum.....	1.500 × 0.384	0.575	67 500	112 600	58.1	15.00	45.2	S. Ang.	0.0	40 sec.	40 sec.	Yield point lost.
	Average of 4..	1.500 × 0.384	0.575	69 000	115 450	60.2	15.94	46.7	S. Ang. / S. Cup.	2.6	3 min. / 40 sec.	3 min. / 40 sec.	
	Maximum....	1.500 × 0.384	0.575	72 000	117 600	62.4	16.75	48.9	S. Ang.	4.3	3 min.	10 sec.	
	Minimum.....	1.494 × 0.502	0.751	60 400	107 300	55.8	15.00	46.5	S. Ang.	2.2	3 min.	3 min.	Yield point lost.
	Average of 4..	1.494 × 0.502	0.751	62 880	111 200	56.5	17.56	47.9	S. Ang. / S. Ang.	3.5	40 sec. / 40 sec.	40 sec. / 40 sec.	
	Maximum.....	1.494 × 0.502	0.751	65 300	113 600	57.5	20.00	50.2	S. Ang.	4.5	3 min.	10 sec.	
	Minimum.....	1.500 × 0.750	1.125	62 600	108 200	56.1	16.50	37.6	S. ½ Cup.	1.9	3 min.	3 min.	Yield point questionable.
	Average of 4..	1.500 × 0.750	1.125	66 150	110 370	59.1	17.56	41.9	S. Cup. / S. ½ Cup.	3.2	40 sec. / 3 min.	40 sec. / 10 sec.	
	Maximum.....	1.500 × 0.750	1.125	68 800	111 500	62.1	18.50	43.6	S. Ang.	4.0	40 sec.	40 sec.	
	Minimum.....	1.242 × 1.000	1.242	58 400	103 400	56.5	20.00	46.4	S. ¾ Cup.	1.5	40 sec.	40 sec.	(a) Cap of Lukens machine not sufficient to break 1-in. piece in 10 sec.
	Average of 4..	1.242 × 1.000	1.242	61 320	104 600	58.6	20.87	49.1	S. ¾ Cup.	2.8	40 sec.	40 sec.	
	Maximum.....	1.242 × 1.000	1.242	63 700	106 500	60.9	22.00	50.6	S. 1 Cup.	4.1	3 min.	3 min.	
6 × 6 × ⅞ in. Angles.—Heat No. 17 749.	Minimum.....	1.500 × 0.757	1.136	64 900	103 000	62.7	16.75	28.2	S. Cup.	1.0	40 sec.	40 sec.	Slightly crystalline.
	Average of 4...	1.500 × 0.757	1.136	66 660	104 370	63.6	17.87	35.2	S. Ang. / Square.	2.9	40 sec. / 40 sec.	40 sec. / 40 sec.	" "
	Maximum.....			68 400	106 100	65.2	19.25	38.5	Irreg. Cup.	4.5	40 sec.	40 sec.	¾ crystalline. ⅛ "
8 × 8 × 1-in. Angles.—Heat No. 17 749.	Minimum.....	1.250 × 1.019	1.273	61 200	99 300	60.4	15.75	37.0	S. ½ Cup.	1.8	40 sec.	40 sec.	
	Average of 4...	1.250 × 1.019	1.273	62 070	100 300	61.8	18.50	41.0	S. ½ Cup. / S. ¾ Cup.	2.6	40 sec. / 40 sec.	40 sec. / 40 sec.	
	Maximum.....	1.250 × 1.019	1.273	63 300	101 200	63.6	21.50	44.0	Irreg. Cup.	4.5	40 sec.	40 sec.	Badly pitted.

Note. This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records.
*Distance from nearest end gauge mark.

TABLE 33.—TENSILE TESTS ON "PARALLEL-SIDED" SPECIMENS OF STRUCTURAL CARBON STEEL.

Made on 200 000-lb. Olsen Machines at Pencoyd Iron Works, Pencoyd, Pa., and at Lukens Iron and Steel Company, Coatesville, Pa.

Test.	Original.		Load per Square Inch.		Elastic ratio. Per cent.	Percentage of elongation in 8 in.	Reduction of area. Per cent.	Fracture.		Speed of Breaking.	
	Section, in inches.	Area, in square inches.	Drop of beam, in pounds.	Ultimate strength, in pounds.				Character.	Location. Inches.*	At drop of beam. 1 in. in:	At break. 1 in. in:
12-in. Universal Plates.—Heat No. 33 342.											
Minimum	1.491 × 0.354	0.528	37 900	62 400	58.6	28.50	53.6	S. ½ Cup.	2.5	15 sec.	15 sec.
Average of 4	1.491 × 0.354	0.528	42 300	65 500	64.4	29.75	56.5	S. ¾ Cup. / S. Cup.	4.4	3 min. / 15 sec.	3 min. / 15 sec.
Maximum	1.491 × 0.354	0.528	48 600	67 600	71.8	31.00	59.5	S. Ang.	5.5	3 min.	10 sec.
Minimum	1.516 × 0.482	0.730	33 600	62 000	51.4	26.25	52.5	S. Ang.	2.0	3 min.	3 min.
Average of 4	1.516 × 0.482	0.730	37 320	64 400	57.9	28.37	55.6	S. Cup. / S. Cup.	3.0	15 sec. / 15 sec.	15 sec. / 15 sec.
Maximum	1.516 × 0.482	0.730	41 400	65 500	63.2	30.50	59.2	S. ¼ Cup.	5.3	3 min.	10 sec.
Minimum	1.491 × 0.756	1.128	30 500	61 100	48.7	32.00	53.6	S. ¾ Cup.	2.8	3 min.	3 min.
Average of 4	1.491 × 0.756	1.128	33 620	62 300	53.9	32.50	56.5	S. Cup. / S. Ang.	3.8	15 sec. / 3 min.	15 sec. / 10 sec.
Maximum	1.491 × 0.756	1.128	36 400	63 300	58.5	33.00	59.1	S. Ang.	4.5	15 sec.	15 sec.
Minimum	1.251 × 0.997	1.247	28 400	58 300	46.1	31.00	56.2	S. ½ Cup.	1.0	15 sec.	15 sec.
Average of 4	1.251 × 0.997	1.247	33 470	60 570	55.2	33.75	58.4	S. Ang. / Irreg. Cup.	3.6	3 min. / 15 sec.	10 sec. / 15 sec.
Maximum	1.251 × 0.997	1.247	39 300	62 100	63.7	38.75	61.4	S. Cup.	5.0	3 min.	3 min.

NOTE. This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records, as shown in the first column.

* Distance from nearest end gauge mark.

TABLE 34.—Tensile Tests of Nickel Steel on Specimens 3 in. Wide, with Holes Punched $\frac{15}{16}$ in. and Sub-Punched $\frac{11}{16}$ in. and Reamed to $\frac{15}{16}$ in. Diameter.

Made on 200 000-lb. Olsen Machine at Drexel Institute.

	Test.	Thickness, in inches.	Dimensions of Original Net Section. Total. Width, in inches.	Dimensions of Original Net Section. Total. Area, in square inches.	Dimensions of Fractured Section. Total. Width, in inches.	Dimensions of Fractured Section. Total. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio, Per cent.	Percentage of Elongation in: 4 in.	Percentage of Elongation in: 8 in.	Reduction of area, Total, Per cent.	Character of fracture.
Heat No. 17 673.	Minimum.....	$\frac{3}{4}$	2.05	0.789	1.89	0.630	60 500	113 800	52.4	5.0	3.0	13.4	S. Ang. / S. $\frac{1}{2}$ Cup.
	Average of 4...	$\frac{3}{4}$	2.06	0.794	1.91	0.660	66 550	114 700	58.0	5.6	3.3	16.9	Silky-Fine Crys.
	Maximum.....	$\frac{3}{4}$	2.08	0.801	1.96	0.694	75 500	115 500	65.0	6.5	3.5	20.5	S. Ang.
	Minimum.....	$\frac{3}{8}$	2.05	0.789	1.97	0.715	66 000	99 600	62.1	2.5	1.5	7.1	Crystalline.
	Average of 4..	$\frac{3}{8}$	2.05	0.790	1.98	0.722	69 270	103 850	66.7	3.4	2.0	8.6	\ L. Crystalline. / R. $\frac{1}{2}$ S., $\frac{1}{2}$ Crys.
	Maximum.....	$\frac{3}{8}$	2.06	0.793	1.99	0.734	72 000	106 300	68.5	4.5	2.5	9.4	\ Crystalline. / L. Silky; R. Crys.
	Minimum.....	$\frac{3}{4}$	2.07	1.553	1.95	1.345	69 000	109 400	61.1	4.0	2.8	11.8	Crystalline.
	Average of 4..	$\frac{3}{4}$	2.07	1.557	1.95	1.356	70 620	111 550	63.3	4.6	3.0	12.8	Crystalline.
	Maximum.....	$\frac{3}{4}$	2.08	1.561	1.97	1.369	72 000	113 700	65.3	5.5	3.3	13.7	Crystalline.
	Minimum.....	$\frac{3}{4}$	2.04	1.530	2.00	1.460	67 000	94 200	70.6	2.0	1.3	1.6	Crystalline.*
	Average of 4...	$\frac{3}{4}$	2.05	1.544	2.01	1.466	71 870	95 220	75.5	2.2	1.3	5.0	Crystalline.*
	Maximum.....	$\frac{3}{4}$	2.07	1.553	2.02	1.475	81 500	97 100	86.5	3.0	1.5	5.5	Crystalline.*
Heat No. 16 080.	Minimum.....	$\frac{3}{4}$	2.05	0.783	1.90	0.652	66 400	95 300	68.0	7.0	3.8	16.4	S. $\frac{1}{2}$ Cup.
	Average of 2...	$\frac{3}{4}$	2.05	0.786	1.91	0.656	66 900	96 450	69.3	7.2	4.1	16.5	S. Ang.
	Maximum.....	$\frac{3}{4}$	2.06	0.789	1.93	0.660	67 400	97 600	70.7	7.5	4.4	16.7	S. Ang
	Minimum.....	$\frac{3}{4}$	2.05	1.497	1.94	1.319	64 900	92 700	63.6	3.5	1.8	7.2	Crystalline
	Average of 2...	$\frac{3}{4}$	2.06	1.509	1.96	1.353	65 850	97 370	67.8	4.0	2.5	10.3	Crystalline.
	Maximum.....	$\frac{3}{4}$	2.07	1.522	1.98	1.388	66 800	102 040	72.1	4.5	3.3	13.4	

* These pieces were bent in punching and were straightened before breaking.

Note.—This table is condensed from the original by giving for each group of tests, minimum, average, and maximum records.

TABLE 35.—TENSILE TESTS—CARBON STEEL—ON SPECIMENS 3 IN. WIDE, WITH HOLES PUNCHED $\frac{15}{16}$ IN. AND SUB-PUNCHED $\frac{11}{16}$ IN. AND REAMED TO $\frac{15}{16}$ IN. DIAMETER.

Made on 200 000-lb. Olsen Machine at Drexel Institute.

Test.	Thickness, in inches.	Dimensions of Original Net Section. Total. Width, in inches.	Dimensions of Original Net Section. Total. Area, in square inches.	Dimensions of Fractured Section. Total. Width, in inches.	Dimensions of Fractured Section. Total. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio. Per cent.	Percentage of Elongation in: 4 in.	Percentage of Elongation in: 8 in.	Reduction of area. Total. Per cent.	Character of fracture.
CARBON STEEL—HEAT NO. 33 342.												
Minimum	$\frac{3}{8}$	2.04	0.734	1.80	0.527	38 000	65 000	58.5	11.0	7.0	25.6	S. Ang.
Average of 4...	$\frac{3}{8}$	2.05	0.738	1.81	0.541	39 500	65 270	60.5	11.5	7.1	26.7	S. Ang.
Maximum	$\frac{3}{8}$	2.06	0.742	1.83	0.547	41 000	65 400	62.7	12.0	7.5	28.6	S. Cup.
Minimum	$\frac{3}{8}$	2.03	0.731	1.85	0.578	40 000	57 560	68.2	8.5	5.5	19.2	S. $\frac{1}{2}$ Cup.
Average of 4...	$\frac{3}{8}$	2.03	0.732	1.86	0.583	40 620	59 420	68.7	8.9	5.7	20.3	S. $\frac{1}{2}$ Cup.
Maximum	$\frac{3}{8}$	2.04	0.734	1.88	0.591	41 000	60 100	69.6	9.5	6.0	21.3	S. Ang.
Minimum	$\frac{3}{4}$	2.04	1.530	1.68	1.014	36 500	65 400	55.1	11.5	7.0	26.9	S. $\frac{1}{4}$-in. near hole. Fine Crystalline.
Average of 4..	$\frac{3}{4}$	2.05	1.540	1.75	1.077	37 250	66 370	56.1	13.0	8.0	30.0	S. Ang.
Maximum	$\frac{3}{4}$	2.07	1.553	1.82	1.119	37 500	68 100	57.3	14.0	8.5	34.1	S. Ang.
Minimum	$\frac{3}{4}$	2.05	1.538	1.96	1.394	41 000	57 000	70.3	3.5	2.5	6.9	Crystalline.*
Average of 4..	$\frac{3}{4}$	2.05	1.540	1.97	1.411	41 500	58 420	71.0	5.0	3.3	8.4	" *
Maximum	$\frac{3}{4}$	2.06	1.546	1.99	1.432	42 500	59 400	71.9	6.0	3.8	9.4	" *

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records, as shown in the first column.

* These pieces were bent in punching, and were straightened before breaking.

TABLE 36.—TENSILE TESTS—NICKEL STEEL AND CARBON STEEL—ON SPECIMENS 3 IN. WIDE, WITH $\frac{15}{16}$-IN. HOLE, PUNCHED-RIVETED.

Made on 200 000-lb. Olsen Machine at Drexel Institute.

Test.	Thickness, in inches.	Dimensions of Original Net Section. Total. Width, in inches.	Dimensions of Original Net Section. Total. Area, in square inches.	Dimensions of Fractured Section. Total. Width, in inches.	Dimensions of Fractured Section. Total. Area, in square inches.	Load per Square Inch. Yield point, in pounds.	Load per Square Inch. Ultimate strength, in pounds.	Elastic ratio. Per cent.	Percentage of Elongation in: 4 in.	Percentage of Elongation in: 8 in.	Reduction of area. Total. Per cent.	Character of fracture.
CARBON STEEL—HEAT NO. 33 342.												
Minimum.....	$\frac{3}{8}$	2.04	0.715	1.91	0.585	41 000	59 200	69.4	9.5	5.3	16.0	S. Ang.
Average of 2...	$\frac{3}{8}$	2.04	0.716	1.91	0.594	41 250	59 300	69.6	10.2	5.8	17.1	½ L. S. Ang.
Maximum.....	$\frac{3}{8}$	2.05	0.718	1.91	0.603	41 500	59 400	69.9	11.0	6.3	18.2	½ R. S. Cup.
Minimum.....	$\frac{3}{4}$	2.05	1.538	2.05	1.522	41 000	48 100	80.2	1.5	1.0	0.5	Coarse Crys.
Average of 4..	$\frac{3}{4}$	2.06	1.546	2.06	1.531	42 620	50 120	85.1	1.8	1.2	0.9	" " *
Maximum.....	$\frac{3}{4}$	2.07	1.553	2.07	1.542	44 000	51 100	88.4	2.0	1.5	1.6	" "
NICKEL STEEL—HEAT NO. 17 673.												
Minimum.....	$\frac{3}{8}$	2.06	0.793	1.99	0.704	75 000	105 900	70.0	4.0	2.0	8.3	Fine Crys.
Average of 2...	$\frac{3}{8}$	2.06	0.793	1.99	0.715	76 500	106 550	71.9	5.0	2.5	9.7	S. Cup.
Maximum.....	$\frac{3}{8}$	2.06	0.793	2.00	0.727	78 000	107 200	73.8	6.0	3.0	11.2	.....
Minimum.....	$\frac{3}{4}$	2.05	1.538	2.03	1.496	74 000	86 300	80.3	1.5	0.7	2.3	Fine Crys.
Average of 4...	$\frac{3}{4}$	2.06	1.547	2.04	1.503	78 500	88 200	89.1	2.0	1.0	2.9	" "
Maximum.....	$\frac{3}{4}$	2.07	1.553	2.05	1.508	82 500	92 300	95.7	3.0	1.5	3.7	" "

NOTE.—This table is condensed from the original by giving, for each group of tests, minimum, average, and maximum records as shown in the first column.

* These pieces were slightly bent in punching, and again in cutting off the rivet heads.

TABLE 37.—Bending Tests on Specimens of Structural Nickel Steel, 3 in. Wide, with Holes Punched $\frac{15}{16}$ in. and Sub-Punched $\frac{11}{16}$ in. and Reamed to $\frac{15}{16}$ in. Diameter, with Holes $\frac{15}{16}$ in. Diameter, Punched-Riveted.

Thickness of material, in inches.	Character of hole.	Angle of bend, in degrees.	2 × (Radius of Bend): Inches.	2 × (Radius of Bend): "t"	Remarks.
12-in. Universal Plates—Heat No. 17 673.					
1 2	Reamed.†	77	1 3 8	2.8	Crack — one side — across — edges not filed.
"	" †	104 (121)	1*	2.0	" both sides — across — edges filed.
"	"	75	1 3 8	2.8	" one side — hole outward — edges not filed.
"	" †	108	13 16	1.6	" one side — across — edges filed.
3 4	"	100	1 1 8	1.5	" both sides — one across.
"	"	89 (96)	1 1 4*	1.7	" both sides — across and through.
1 2	Punched.†	57	1 15 16	3.9	" both sides — hole outward.
"	" †	57	2	4.0	" " " " "
"	" †	65	2 1 8	4.3	" " " one across.
"	"	58	1 3 4	3.5	" " " hole outward.
3 4	"	38 (46)	4*	5.3	" " " across and through.
"	"	39 (47)	3 7 8*	5.2	" " " " " "
1 2	Punched-riveted.†	54	2 1/8	4.3	" " " hole outward.
"	" †	33	4 11 16	9.4	" one " across and through.
"	" †	54	2 1 8	4.3	" both " hole outward.
3 4	"	29 (34)	5 3/4*	7.7	" " " across and through.
"	"	27 (34)	5 13 16*	7.7	" " " " " "

"t" = Thickness of piece.
* Angle and radius of bend were estimated to be those existing when crack first started in piece.
† These pieces had been stressed to 32 000 lb. per sq. in. before being bent.

TABLE 38.—Bending Tests on Specimens of Medium Carbon Steel, 3 in. Wide, with Holes Punched $\frac{15}{16}$ in. and Sub-Punched $\frac{11}{16}$ in. and Reamed to $\frac{15}{16}$ in. Diameter, with Holes $\frac{15}{16}$ in. Diameter, Punched-Riveted.

Thickness of material, in inches.	Character of hole.	Angle of bend, in degrees.	2 × (Radius of Bend): Inches.	2 × (Radius of Bend): "t"	Opening at 1 in. from inside edge, in inches.	Remarks.
12-in. Universal Plates—Heat No. 33 342.						
$\frac{1}{2}$	Reamed	190	7 16	0.9	11 16	Crack—both sides, hole—outward—edges not filed.
$\frac{1}{2}$	"	193	7 16	0.9	13 16	" " " " " " "
$\frac{1}{2}$	"	190	7 16	0.9	5 8	" " " " " edges filed.
$\frac{1}{2}$	"	198	11 16	1.4	1 1 16	" " " " " edges not filed.
$\frac{1}{2}$	"	198	13 16	1.1	1 3 16	" " " " "
$\frac{3}{4}$	"	193	3 4	1.0	1 3 8	" " " " "
$\frac{3}{4}$	"	191	11 16	1.4	1 1 4	" " " " "
$\frac{1}{2}$	Punched	183	3 4	1.5	1 7 16	" " " " "
$\frac{1}{2}$	"	179	7 8	1.8	1 9 16	" " " " "
$\frac{1}{2}$	"	196	5 8	1.3	1 1 4	" " " " "
$\frac{3}{4}$	"	86	1 1 2	2.0	.....	" " " " "
$\frac{3}{4}$	"	82	1 7 8	2.5	.....	" " " " "
$\frac{1}{2}$	Punched-riveted	172	1	2.0	1 3 4	" " " " "
$\frac{1}{2}$	"	175	7 8	1.8	1 7/16	" " " " "
$\frac{1}{2}$	"	184	15 16	1.9	1 1 2	" " " " "
$\frac{1}{2}$	"	169	15 16	1.9	1 13 16	" " " " "
$\frac{3}{4}$	"	60(69)*	2 5 8	3.5	.....	" " " across and through.
$\frac{3}{4}$	"	64	2 3 8	3.2	.....	" " " hole outward.

"t" = Thickness of piece.

* Angle and radius of bend were estimated to be those existing when crack first started in piece.

TABLE 39.—Drifting Tests: Of Structural Nickel Steel and Carbon Steel, According to Specifications.

Mark.	Diameter. Original, in inches.	Diameter. Enlarged. Specified, in inches.	Diameter. Enlarged. Actual, in inches.	Enlargement. Specified. Per cent.	Enlargement. Actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks. All holes punched full size.
Nickel Steel.—12 by ⅜-in. Universal Plate.—Heat No. 17 673. Mark *DS* 20.								
A	13/16	1	1 1/32	25	27	1 5/8	Sheared.	No cracks—hole or edge—either side.
B	15/16	1 3/16	1 3/16	25	27	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 3/16	30	27	1 13/32	Rolled.	" " " " " " "
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	" " " " " " "
E	13/16	1 1/16	1 3/32	33	35	1 5/8	"	" " " " " " "
F	13/16	1 1/8	1 1/8	38	38	1 1/2	Rolled.	" " " " " " "
G	13/16	1 7/32	1 3/16	50	46	1 5/8	Sheared.	" " " " " " "
H	15/16	1 13/32	1 3/8	50	47	1 7/8	"	Fair-sized crack at hole, die side only.
I	15/16	1 13/32	1 5/16	50	40	1 1/2	"	Crack ½ in. long from edge of plate; no cracks at hole.
J	15/16	1 13/32	1 11/32	50	43	1 1/2	Rolled.	Four fair-sized cracks at hole, die side only.
K	13/16	1 1/2	1 15/32	85	81	1	"	Four large cracks at hole, punch side only.
L	13/16	1 1/2	1 7/32	85	50	1 1/4	Sheared.	Crack ½ in. long from edge of plate; no cracks at hole.

Note.—Two interior holes in nickel steel marked *C* were drifted to 1 3/16 in. in diameter, the inside hole had one slight crack.

Carbon Steel.—12 by ⅜-in. Universal Plate.—Heat No. 33 342. Mark *CDS* 20.								
A	13/16	1	1 1/16	25	31	1 5/8	Sheared.	No cracks, hole or edge—either side.
B	15/16	1 3/16	1 5/16	25	40	1 7/8	"	" " " " " " "
C	15/16	1 7/32	1 5/32	30	23	1 13/32	Rolled.	Four slight cracks at hole—die side only.
D	15/16	1 1/4	1 7/32	33	30	1 7/8	Sheared.	No cracks.
E	13/16	1 1/16	1 1/16	33	31	1 5/8	"	" "
F	13/16	1 1/8	1 1/4	38	54	1 1/2	Rolled.	" "
G	13/16	1 7/32	1 3/16	50	48	1 5/8	Sheared.	" "
H	15/16	1 13/32	1 13/32	50	50	1 7/8	"	Small crack at hole—die side only.
I	15/16	1 13/32	1 7/16	50	53	1 1/2	"	No crack.
J	15/16	1 13/32	1 13/32	50	50	1 1/2	Rolled.	" "
K	13/16	1 1/2	1 1/2	85	85	1	"	Small cracks at hole, die side only.
L	13/16	1 1/2	1 3/8	85	69	1 1/4	Sheared.	Small cracks at hole, die side only. / Crack ¾ in. long from edge of plate.

Note.—Two interior holes in carbon steel marked *C* were drifted to 1 3/32 in. and 1 1/8 in. in diameter, respectively. Slight cracks, four in number, were found on one side where bulging was excessive.

TABLE 40.—Drifting Tests: Structural Nickel Steel and Carbon Steel, According to Specifications.

Mark.	Diameter. Original, in inches.	Diameter. Enlarged. Specified, in inches.	Diameter. Enlarged. Actual, in inches.	Enlargement. Specified. Per cent.	Enlargement. Actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks. All holes punched full size.
Nickel Steel.—12 by ¾ in. Universal Plate.—Heat No. 17 673. Mark *DS* 113.								
A	13 16	1	1	25	25	1 5 8	Sheared.	No cracks, hole or edge, either side.
B	15 16	1 3 16	1 5 32	25	23	1 7 8	"	" " " " " " " "
C	15 16	1 7 32	1 7 32	30	30	1 13 32	Rolled.	" " " " " " " "
D	15 16	1 1 4	1 1 4	33	33	1 7 8	Sheared.	" " " " " " " "
E	13 16	1 1 16	1 1 64	33	25	1 5 8	"	" " " " " " " "
F	13 16	1 1 8	1 1 8	38	38	1 1 2	Rolled.	One fair-sized crack at hole, die side only.
G	13 16	1 7 32	1 3 16	50	46	1 5 8	Sheared.	No cracks.
H	15 16	1 13 32	1 3 8	70	47	1 7 8	"	One fair-sized crack at hole, punch side only.
I	15 16	1 13 32	1 7 32	50	30	1 1 2	"	Crack $\frac{1}{2}$ in. long from edge of plate; no cracks at hole.
J	15 16	1 13 32	1 11 32	50	43	1 1 2	Rolled.	No cracks.
K	13 16	1 1 2	1 1 2	85	85	1	"	Three large cracks at hole, punch side only.
L	13 16	1 1 2	1 3 16	85	46	1 1 4	Sheared.	Crack $\frac{1}{8}$ in. long from edge of plate. No cracks at hole.

Note.—Two interior holes marked *C* were drifted to 1 7 32 in. in diameter; the middle hole had several fine cracks.

Carbon Steel.—12 by ¾-in. Universal Plate.—Heat No. 33 342. Mark *CDS* 113.								
A	13 16	1	1	25	25	1 5 8	Sheared.	No cracks, hole or edge, either side.
B	15 16	1 3 16	1 9 32	25	37	1 7 8	"	" " " " " " " "
C	15 16	1 7 32	1 7 32	30	30	1 13 32	Rolled.	" " " " " " " "
D	15 16	1 1 4	1 1 4	33	33	1 7 8	Sheared.	" " " " " " " "
E	13 16	1 1 16	1 3 32	33	35	1 5 8	"	" " " " " " " "
F	13 16	1 1 8	1 1 8	38	38	1 1 2	Rolled.	Several small cracks at hole, die side only.
G	13 16	1 7 32	1 1 4	50	54	1 5 8	Sheared.	No cracks.
H	15 16	1 13 32	1 7 16	50	53	1 7 8	"	" "
I	15 16	1 13 32	1 5 16	50	40	1 1 2	"	Crack $\frac{1}{2}$ in. long from edge of plate; no cracks at hole.
J	15 16	1 13 32	1 3 8	50	47	1 1 2	Rolled.	Several small cracks at hole, die side only.
K	13 16	1 1 2	1 7 16	85	77	1	"	Four large cracks at hole, die side only.
L	13 16	1 1 2	1 1 2	85	85	1 1 4	Sheared.	Small crack at edge in corner only; no cracks at hole.

Note. Two interior holes marked *C* were drifted to 1 7 32 in. in diameter; each had numerous small cracks.

TABLE 41.—Drifting Tests: Structural Nickel Steel and Carbon Steel, According to Specifications.

Mark.	Diameter. Original, in inches.	Diameter. Enlarged. Specified, in inches.	Diameter. Enlarged. Actual, in inches.	Enlargement. Specified. Per cent.	Enlargement. Actual. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks. All drilled from the solid.
Nickel Steel.—12 by 1-in. Universal Plate.—Heat No. 17 673. Mark *DS* 164.								
A	13 16	1	1 1 64	25	25	1 5 8	Sheared.	No cracks, hole or edge, either side.
B	15 16	1 3 16	1 7 32	25	30	1 7 8	"	" " " " " " "
C	15 16	1 7 32	1 1 2	30	60	1 13 32	Rolled.	" " " " " " "
D	15 16	1 1 4	1 7 32	33	30	1 7 8	Sheared.	" " " " " " "
E	13 16	1 1 16	1 1 8	33	38	1 5 8	"	" " " " " " "
F	13 16	1 1 8	1 3 32	38	35	1 1 2	Rolled.	" " " " " " "
G	13 16	1 7 32	1 11 64	50	44	1 5/8	Sheared.	" " " " " " "
H	15 16	1 13 32	1 11 32	50	43	1 7 8	"	" " " " " " "
I	15 16	1 13 32	1 1/4	50	33	1 1 2	"	Break from edge to hole; no other cracks.
J	15 16	1 13 32	1 3 8	50	47	1 1 2	Rolled.	No cracks.
K	13 16	1 1 2	1 7 32	85	50*	1	"	" "
L	13 16	1 1 2	1 1 8	85	38	1 1 4	Sheared.	Break from edge to hole; no other cracks.

Note.—Two interior holes marked *C* were drifted to 1 7 32 in. in diameter; no cracks.

Carbon Steel.—12 by 1-in. Universal Plate.—Heat No. 33 342. Mark *CDS* 164.								
A	13 16	1	1	25	25	1 5 8	Sheared.	No cracks, hole or edge, either side.
B	15 16	1 3/16	1 1/4	25	33	1 7 8	"	" " " " " " "
C	15/16	1 7 32	1 1 4	30	33	1 13 32	Rolled.	" " " " " " "
D	15 16	1 1 4	1 7 32	33	30	1 7 8	Sheared.	" " " " " " "
E	13 16	1 1/16	1 1 16	33	31	1 5 8	"	" " " " " " "
F	13 16	1 1 8	1 1 8	38	38	1 1 2	Rolled.	" " " " " " "
G	13/16	1 7 32	1 3 16	50	46	1 5/8	Sheared.	Slight crack at corner of edge—one side; no cracks at hole.
H	15 16	1 13 32	1 13 32	50	50	1 7 8	"	Crack ½ in. long from edge of plate; no cracks at hole.
I	15 16	1 13 32	1 5 16	50	40	1 1/2	"	Break from edge to hole; no other cracks.
J	15 16	1 13 32	1 11 32	50	43	1 1/2	Rolled.	No cracks.
K	13 16	1 1/2	1 9 16	85	93	1	"	Four large ones at hole, punch side; small ones, die side.
L	13/16	1 1/2	1 1/2	85	85	1 1/4	Sheared.	Crack ⅛ in. long from edge of plate; no cracks at hole.

Note.—Two interior holes marked *C* were drifted to 1 1 4 in. in diameter; no cracks.
*Through an oversight, this hole was not enlarged further.

TABLE 42.—Drifting Tests: Structural Nickel Steel and Carbon Steel. Holes Drifted to Cracking.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per cent.	Distance center of hole to edge, in inches.	Character of edge.	Remarks.
Nickel Steel.—12 by ⅝-in. Universal Plate.—Heat No. 17 673.—Mark *DC* 21.							
M	Punched	15/16	1 15/32	57	1 1/2	Sheared.	Break—edge to hole. Large crack at hole, punch side only.
N	Drilled.	15/16	1 1/4	33	1 1/2	"	Crack ¼ in. long from edge of plate; no cracks at hole.
O	Reamed.	15/16	1 1/2	60	1 7/8	Rolled.	Three cracks (one large) at hole, punch side only.
P	Punched.	15/16	1 7/16	53	....	Interior.	One large crack at hole, punch side; two large cracks, die side.
Q	"	15/16	1 3/8	47	1 7/8	Rolled.	One small crack at hole, die side only.
R	"	15/16	1 13/32	50	1 7/8	"	Four large cracks at hole, die side only.
S	Drilled.	15/16	1 7/16	53	....	Interior.	Several large cracks at hole, punch side; one crack, die side.
T	"	15/16	1 1/2	60	1 7/8	Rolled.	One large crack at hole, punch side; two cracks, die side.
U	Reamed.	15/16	1 3/8	47	1 7/8	Sheared.	Crack ½ in. long from edge of plate; no cracks at hole.
V	Punched.	15/16	1 13/32	50	1 7/8	"	Crack ¾ in. long from edge of plate; no cracks at hole.
Carbon Steel.—12 by ⅝-in. Universal Plate.—Heat No. 33 342.—Mark *CDS* 21.							
M	Punched.	15/16	1 5/8	73	1 1/2	Sheared.	Crack 1 in. long from edge of plate; no cracks at hole.
N	Drilled.	15/16	1 9/16	67	1 1/2	"	Crack ¼ in. long from edge of plate; six cracks at hole, die side.
O	Reamed.	15/16	1 23/32	83	1 7/8	Rolled.	Several large cracks at hole, die side.
P	Punched.	15/16	1 25/32	90	....	Interior.	Several small cracks at hole, die side.
Q	"	15/16	1 31/32	110	1 7/8	Rolled.	Small cracks at hole, punch side; two large cracks, die side.
R	"	15/16	2 1/32	117	1 7/8	"	Badly cracked at hole, both sides.
S	Drilled.	15/16	1 13/16	93	....	Interior.	Badly cracked at hole, die side.
T	"	15/16	1 7/8	100	1 7/8	Rolled.	Badly cracked at hole, die side.
U	Reamed.	15/16	1 5/8	73	1 7/8	Sheared.	Crack ½ in. long from edge of plate; cracks at hole, each side.
V	Punched.	15/16	1 5/8	73	1 7/8	"	Crack ¾ in. long from edge of plate; small cracks at hole, punch side.

TABLE 43.—Drifting Tests: Structural Nickel Steel and Carbon Steel, Holes Drifted to Cracking.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per Cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
Nickel Steel.—12 by ¾-in. Universal Plate.—Heat No. 17 673—Mark *DC* 114.							
M.....	Punched.	15/16	1 7/16	53	1 1/2	Sheared.	Crack 1/8 in. long from edge of plate; no cracks at hole.
N.....	Drilled.	15/16	1 1/4	33	1 1/2	"	Break from edge to hole; no other cracks at hole.
O.....	Reamed.	15/16	1 3/4	87	1 7/8	Rolled.	Two small cracks at hole, punch side; four large cracks, die side.
P.....	Punched.	15/16	1 5/8	73	.....	Interior.	One large crack at hole, each side.
Q.....	"	15/16	1 11/16	80	1 7/8	Rolled.	Three small cracks at hole, each side.
R.....	"	15/16	1 3/4	87	1 7/8	"	Three fair-sized cracks at hole, each side.
S.....	Drilled.	15/16	1 5/8	73	.....	Interior.	Four small cracks at hole, punch side; two large cracks, die side.
T.....	"	15/16	1 5/8	73	1 7/8	Rolled.	Three small cracks at hole, punch side; one large crack, die side.
U.....	Reamed.	15/16	1 5/8	73	1 7/8	Sheared.	Break from edge to hole; one other small crack at hole, each side.
V.....	Punched.	15/16	1 1/2	60	1 7/8	"	Crack 1 in. long from edge to hole; other small cracks, each side.
Carbon Steel.—12 by ¾-in. Universal Plate.—Heat No. 33 342—Mark *CDC* 114.							
M.....	Punched.	15/16	1 11/16	80	1 1/2	Sheared.	Crack 1/2 in. long from edge to hole; one small crack at hole, die side.
N.....	Drilled.	15/16	1 11/16	80	1 1/2	"	Crack 1/4 in. long from edge to hole; no cracks at hole.
O.....	Reamed.	15/16	1 3/4	87	1 7/8	Rolled.	Three cracks at hole, die side.
P.....	Punched.	15/16	2 3/16	133	.....	Interior.	Six large cracks at hole, punch side, three cracks, die side.
Q.....	"	15/16	2 3/16	133	1 7/8	Rolled.	Ten large cracks at hole, die side; two cracks, punch side.
R.....	"	15/16	1 9/16	67	1 7/8	"	Four large cracks at hole, die side.
S.....	Drilled.	15/16	1 13/16	93	.....	Interior.	One large crack at hole, punch side.
T.....	"	15/16	2	113	1 7/8	Rolled.	Four cracks at hole, punch side; two, die side.
U.....	Reamed.	15/16	1 1/2	60	1 7/8	Sheared.	Crack 1/4 in. long from edge to hole; no cracks at hole.
V.....	Punched.	15/16	1 9/16	67	1 7/8	"	Crack 1/2 in. long from edge to hole; one small crack at hole, each side.

TABLE 44.—Drifting Tests: Structural Nickel Steel and Carbon Steel, Holes Drifted to Cracking.

Mark.	Character of hole.	Diameter. Original, in inches.	Diameter. Enlarged, in inches.	Enlargement. Per cent.	Distance, center of hole to edge, in inches.	Character of edge.	Remarks.
Nickel Steel.—12 by 1-in. Universal Plate.—Heat No. 17 673.—Mark *DC* 165.							
M	Drilled.	15/16	1 5/16	40	1 1/2	Sheared.	Crack 1/4 in. long from edge to hole; no cracks at hole.
N	"	15/16	1 11/32	43	1 1/2	"	Crack 1/4 in. long from edge to hole; no cracks at hole.
O	"	15/16	1 5/8	73	1 7/8	Rolled.	Two large cracks at hole, punch side; three, die side.
P	"	15/16	1 5/8	73	.....	Interior.	Fine cracks at hole, punch side; six large cracks, die side.
Q	"	15/16	1 13/16	93	1 7/8	Rolled.	Five cracks (one large) at hole, punch side; two, die side.
R	"	15/16	1 9/16	67	1 7/8	"	Two large cracks at hole, punch side; no cracks die side.
S	"	15/16	1 11/16	80	.....	Interior.	Four small cracks at hole, punch side; large cracks, die side.
T	"	15/16	1 13/16	93	1 7/8	Rolled.	Several cracks (one large) at hole, punch side; one, die side.
U	"	15/16	1 3/16	53	1 7/8	Sheared.	Break from edge to hole, two other cracks at hole, punch side.
V	"	15/16	1 7/16	53	1 7/8	"	Crack 1/2 in. long from edge to hole; one crack at hole, die side.
Carbon Steel.—12 by 1-in. Universal Plate.—Mark *CDS* 165.							
M	Drilled.	15/16	1 7/8	100	1 1/2	Sheared.	Crack 1/4 in. long from edge to hole; one crack at hole, die side.
N	"	15/16	1 9/16	67	1 1/2	"	Crack 1/4 in. long from edge to hole; no cracks at hole.
O	"	15/16	2	113	1 7/8	Rolled.	Four large cracks at hole, punch side; four small cracks, die side.
P	"	15/16	2	113	.....	Interior.	Four large cracks at hole, die side; small crack, punch side.
Q	"	15/16	2	113	1 7/8	Rolled.	Seven large cracks at hole, die side.
R	"	15/16	2	113	1 7/8	"	Six large cracks at hole, die side; one crack, punch side.
S	"	15/16	2 1/8	127	.....	Interior.	Several cracks at hole, each side.
T	"	15/16	2 1/16	120	1 7/8	Rolled.	Five large cracks at hole, die side; one small crack, punch side.
U	"	15/16	1 9/16	67	1 7/8	Sheared.	Crack 1/4 in. long from edge to hole; fine cracks at hole, punch side.
V	"	15/16	1 11/16	80	1 7/8	"	Crack 1/2 in. long from edge to hole; no cracks at hole.

TABLE 45.—"Bearing-on-Pins" Tests: To Determine Amount of Yielding of Pin and Supports.

Readings taken with Olsen's Compression Micrometer.

Load.	Micrometer Readings, in Inches.								
	Pin No. 1.				Pin No. 2.				
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0018	0.0014	0.0009	0.0011	0.0017	0.0014	0.0016	0.0008	0.0011
10 000	0.0035	0.0023	0.0019	0.0022	0.0031	0.0026	0.0029	0.0017	0.0028
*15 000	0.0047	0.0033	0.0027	0.0029	0.0037	0.0030	0.0034	0.0022	0.0035
20 000	0.0052	0.0040	0.0034	0.0036	0.0042	0.0033	0.0039	0.0025	0.0040
*25 000	0.0064	0.0047	0.0037	0.0042	0.0048	0.0039	0.0044	0.0026	0.0042
30 000	0.0072	0.0053	0.0040	0.0048	0.0054	0.0044	0.0049	0.0029	0.0048
*35 000	0.0077	0.0058	0.0045	0.0050	0.0059	0.0047	0.0051	0.0032	0.0052
40 000	0.0081	0.0063	0.0047	0.0053	0.0065	0.0050	0.0054	0.0035	0.0054
*45 000	0.0086	0.0068	0.0050	0.0054	0.0070	0.0052	0.0056	0.0038	0.0057
50 000	0.0089	0.0073	0.0053	0.0058	0.0074	0.0056	0.0060	0.0041	0.0061
*55 000	0.0092	0.0077	0.0056	0.0061	0.0076	0.0059	0.0061	0.0042	0.0063
60 000	0.0095	0.0081	0.0061	0.0064	0.0080	0.0062	0.0063	0.0045	0.0066
*65 000	0.0099	0.0084	0.0061	0.0065	0.0083	0.0066	0.0065	0.0047	0.0068
70 000	0.0103	0.0088	0.0066	0.0067	0.0087	0.0070	0.0068	0.0049	0.0071
*75 000	0.0106	0.0093	0.0068	0.0068	0.0097	0.0073	0.0070	0.0952	0.0075
80 000	0.0111	0.0096	0.0071	0.0071	0.0105	0.0075	0.0072	0.0053	0.0078
*85 000	0.0116	0.0101	0.0075	0.0074	0.0110	.....	.....	.....	.....
90 000	0.0121	0.0106	0.0079	0.0076	0.0118	.....	.....	.....	.....
*95 000	0.0125	0.0110	0.0083	0.0078	.....	.....	.....	.....	.....
100 000	.....	0.0117	0.0089	0.0081	.....	.....	.....	.....	.....

Note.—This table is condensed from the original.
*Interpolated values in horizontal lines marked in this way.

TABLE 46.—"Bearing-on-Pins" Tests: Structural Plate Material.

Readings taken with Olsen's Compression Micrometer.

Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	Micrometer Readings, in Inches.					
	Nickel Steel—Heat No. 17 673.			Carbon Steel—Heat No. 33 342.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0017	0.0010	0.0010	0.0011	0.0013	0.0017
10 000	0.0030	0.0018	0.0020	0.0025	0.0023	0.0028
*15 000	0.0041	0.0027	0.0031	0.0035	0.0033	0.0041
20 000	0.0050	0.0037	0.0040	0.0045	0.0042	0.0052
*25 000	0.0057	0.0045	0.0048	0.0055	0.0052	0.0063
30 000	0.0064	0.0052	0.0054	0.0067	0.0066	0.0076
*35 000	0.0072	0.0060	0.0060	0.0083	0.0086	0.0098
40 000	0.0082	0.0067	0.0067	0.0111	0.0118	0.0124
*45 000	0.0091	0.0077	0.0073	0.0143	0.0156	0.0158
50 000	0.0101	0.0087	0.0083	0.0193	0.0206	0.0206
*55 000	0.0112	0.0100	0.0095	0.0255	0.0271	0.0265
60 000	0.0122	0.0120	0.0108	0.0333	0.0359	0.0346
*65 000	0.0135	0.0129	0.0121	0.0432	0.0477	0.0450
70 000	0.0148	0.0142	0.0135	0.0551	0.0625	0.0563
*75 000	0.0163	0.0151	0.0148	0.0696	0.0803	0.0693
80 000	0.0182	0.0167	0.0167	0.0850	0.0997	0.0871
*85 000	0.0204	0.0188	0.0188	0.1031	0.1201	0.1074
90 000	0.0229	0.0212	0.0210	0.1212	0.1420	0.1290
*95 000	0.0256	0.0238	0.0236	.....	.....	.....
100 000	0.0288	0.0263	0.0265	.....	.....	.....

Note.—This table is condensed from the original.

* Interpolated values in horizontal lines marked in this way.

TABLE 47.—"Bearing-on-Pins" Tests: Structural Plate Material.
Amount of Compression in Test Pieces.
Crrected for Yielding of Pin and Supports.
Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	Amount of Compression, in Inches.					
	Nickel Steel—Heat No. 17 673.			Carbon Steel—Heat No. 33 342.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0004	—0.0002	—0.0001	—0.0003	—0.0001	0.0002
10 000	0.0007	—0.0004	—0.0002	0.0002	0.0000	0.0005
*15 000	0.0009	—0.0004	0.0001	0.0002	0.0000	0.0008
*20 000	0.0011	—0.0001	0.0003	0.0005	0.0002	0.0012
*25 000	0.0011	0.0001	0.0006	0.0008	0.0005	0.0016
30 000	0.0013	0.0003	0.0007	0.0014	0.0013	0.0023
*35 000	0.0016	0.0007	0.0010	0.0025	0.0028	0.0040
40 000	0.0022	0 0010	0.0013	0.0048	0.0055	0.0061
*45 000	0.0026	0.0017	0.0017	0.0075	0.0088	0.0089
50 000	0.0032	0.0022	0.0022	0.0120	0.0133	0.0133
*55 000	0.0039	0.0032	0.0031	0.0178	0.0194	0.0188
60 000	0.0045	0.0048	0.0041	0.0252	0.0278	0.0265
*65 000	0.0055	0.0055	0.0051	0.0348	0.0393	0.0366
70 000	0.0065	0.0064	0.0062	0.0463	0.0537	0.0475
*75 000	0.0076	0.0070	0.0072	0.0603	0.0710	0.0600
80 000	0.0092	0.0083	0.0089	0.0754	0.0901	0.0775
*85 000	0.0109	0.0100	0.0106	0.0930	0.1100	0.0973
90 000	0.0130	0.0120	0.0125	0.1106	0.1314	0.1184
*95 000	0.0153	0.0142	0.0147	.....	.....	.....
100 000	0.0179	0.0162	0.0172	.....	.....	.....

Note.—This table is condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 48.—"Bearing-on-Pins" Tests: Plate Rivet Material.
Readings taken with Olsen's Compression Micrometer.
Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	Micrometer Readings, in Inches.					
	Nickel steel—Heat No. 2 096.			Carbon steel—Heat No. 19 241.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0018	0.0011	0.0014	0.0017	0.0011	0.0015
10 000	0.0034	0.0021	0.0033	0.0033	0.0024	0.0029
*15 000	0.0044	0.0031	0.0045	0.0048	0.0037	0.0044
20 000	0.0053	0.0039	0.0054	0.0064	0.0053	0.0061
*25 000	0.0060	0.0046	0.0063	0.0079	0.0068	0.0079
30 000	0.0066	0.0054	0.0072	0.0115	0.0100	0.0108
*35 000	0.0075	0.0063	0.0082	0.0160	0.0143	0.0148
40 000	0.0083	0.0072	0.0096	0.0206	0.0190	0.0194
*45 000	0.0197	0.0079	0.0115	0.0266	0.0247	0.0238
50 000	0.0114	0.0092	0.0132	0.0340	0.0313	0.0290
*55 000	0.0135	0.0107	0.0154	0.0482	0.0431	0.0364
60 000	0.0158	0.0128	0.0177	0.0635	0.0568	0.0487
*65 000	0.0181	0.0150	0.0202	0.0800	0.0732	0.0612
70 000	0.0205	0.0179	0.0231	0.1003	0.0885	0.0785
*75 000	0.0234	0.0217	0.0264	.....	.....	.....
80 000	0.0271	0.0259	0.0298	.....	.....	.....
*85 000	0.0308	.....	.....	.....	.....	.....
90 000	0.0359	.....	.....	.....	.....	.....

Note.—This table condensed from the original.
* Interpolated values in horizontal lines marked in this way.

TABLE 49.—"Bearing-on-Pins" Tests: Plate Rivet Material.
Amount of Compression in Test Pieces.
Corrected for Yielding on Pin and Supports.
Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Load.	Amount of Compression, in Inches.					
	Nickel Steel — Heat No. 2 096.			Carbon Steel — Heat No. 19 241.		
2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
*5 000	0.0003	0.0000	0.0005	0.0002	—0.0002	0.0005
10 000	0.0006	0.0001	0.0013	0.0005	—0.0001	0.0005
*15 000	0.0011	0.0006	0.0019	0.0016	0.0009	0.0013
20 000	0.0017	0.0010	0.0024	0.0028	0.0029	0.0026
*25 000	0.0018	0.0015	0.0031	0.0037	0.0031	0.0042
30 000	0.0019	0.0019	0.0037	0.0068	0.0059	0.0067
*35 000	0.0025	0.0025	0.0044	0.0110	0.0098	0.0102
40 000	0.0031	0.0031	0.0054	0.0154	0.0143	0.0146
*45 000	0.0042	0.0035	0.0069	0.0211	0.0197	0.0187
50 000	0.0056	0.0045	0.0084	0.0282	0.0260	0.0236
*55 000	0.0075	0.0059	0.0104	0.0422	0.0377	0.0307
60 000	0.0095	0.0077	0.0125	0.0572	0.0511	0.0428
*65 000	0.0115	0.0097	0.0148	0.0734	0.0673	0.0550
70 000	0.0136	0.0124	0.0174	0.0934	0.0823	0.0721
*75 000	0.0162	0.0159	0.0204	.....	.....	.....
80 000	0.0197	0.0199	0.0236	.....	.....	.....

Note.—This table is condensed from the original.
* Interpolated values on horizontal lines marked in this way.

TABLE 50. "BEARING-ON-PINS" TESTS: NICKEL STEEL AND CARBON STEEL.

Made on 100 000-lb. Olsen Machine; Readings Taken by Olsen's Compression Micrometer, Reading Directly to Thousandths of an Inch. Diameter of Pin = 1 in. Length of Piece above Pin = 3.80 in.

Material.	Thickness, in inches.	Elastic limit, in pounds per square inch.	Amount of Compression, in Inches, Under Load, per Square Inch of:					Dimensions of Pin-Hole:		Increased thickness of metal over pin-hole, in inches.
			Elastic limit.	55 000	75 000	95 000	115 000	Original, in inches.	Distorted, in inches.	
Plate Carbon, 33 342	0.750	43 300	0.0021	0.0056	0.0196	0.0488	0.0972	1.00 by 1.00	1.09 by 1.03	0.08
	0.750	40 000	0.0014	0.0062	0.0210	0.0578	0.1156	1.00 " 1.00	1.12 " 1.04	0.07
	0.750	38 300	0.0020	0.0067	0.0204	0.0500	0.1026	1.00 " 1.00	1.10 " 1.04	0.06
Plate nickel, 17 673	0.750	.....	.....	0.0023	0.0040	0.0067	0.0114	1.00 " 1.00	1.02 " 1.01	0.00
	0.750	69 300	0.0024	0.0012	0.0036	0.0064	0.0105	1.00 " 1.00	1.02 " 1.01	0.01
	0.750	66 700	0.0022	0.0014	0.0034	0.0064	0.0110	1.00 " 1.00	1.02 " 1.01	0.02
Rivet carbon, 19 241	0.765	32 700	0.0036	0.0184	0.0490	0.1070	.....	1.02 " 1.01	1.10 " 1.03	0.05
	0.765	31 400	0.0027	0.0174	0.0450	0.0950	.....	1.02 " 1.02	1.10 " 1.04	0.05
	0.765	31 400	0.0039	0.0168	0.0366	0.0855	.....	1.02 " 1.02	1.09 " 1.03	0.05
Rivet nickel, 2 096	0.710	59 200	0.0034	0.0030	0.0069	0.0124	0.0214	1.02 " 1.01	1.04 " 1.01	0.01
	0.710	62 000	0.0034	0.0029	0.0033	0.0112	0.0214	1.02 " 1.02	1.03 " 1.02	0.01
	0.710	53 500	0.0048	0.0050	0.0098	0.0160	0.0250	1.02 " 1.02	1.04 " 1.03	0.01
AVERAGES.										
Plate carbon	0.750	41 200	0.0018	0.0062	0.0203	0.0522	0.1051	1.00 by 1.00	1.10 by 1.04	0.07
Plate nickel	0.750	68 300	0.0023	0.0016	0.0037	0.0065	0.0109	1.00 " 1.00	1.02 " 1.01	0.01
Rivet carbon	0.765	31 800	0.0034	0.0175	0.0435	0.0960	.....	1.02 " 1.02	1.10 " 1.03	0.05
Rivet nickel	0.710	58 200	0.0039	0.0036	0.0067	0.0132	0.0226	1.02 " 1.02	1.04 " 1.02	0.01

PERCENTAGE THAT ABOVE AVERAGE VALUES FOR NICKEL STEEL ARE TO CARBON STEEL.

Plate material	166.	128.	26.	18.	12.	10.
Rivet material	183.	115.	21.	15.	14.	...

TABLE 51.—(*Continued*).

44 400	0.120		0.135							
45 400	0.125									
46 300	0.128		0.143							
47 200	0.130		0.143			0.001				
48 100	0.135		0.150			0.003				
49 000			0.153			0.004				
49 900	0.138		0.155	0.001		0.094				
50 900	0.143			0.001						
51 800	0.145		0.160	0.001		0.005				
52 800	0.148			0.003						
53 700	0.155		0.170	0.005		0.008				
54 600	0.155			0.005						
55 500	0.163		0.175	0.008		0.013				
56 400	0.163			0.009						
57 300	0.168			0.010						
58 500	0.173			0.013						
60 500	0.180			0.015						
61 900	0.190			0.020						
68 500				Failed.	0.33 Failed.					
69 200						Failed.				
Order of testing	5	1	6				2	3	4	

TABLE 51.—Compression Tests of Struts, 10 ft. Long: Structural Nickel Steel and Medium Carbon Steel.

Made on 2 160 000-lb. Hydraulic Testing Machine, Phœnix Iron Company.

Load, in pounds square inch.	Nickel Steel.						Carbon Steel.					
	Temporary shortening, in inches.			Permanent set, in inches.			Temporary shortening, in inches.			Permanent set, in inches.		
	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.
11 000	0.023			0.000	0.000	0.000		0.020	0.038	0.000	0.000	0.000
12 000			0.030									
14 800							0.025					
18 500	0.045		0.048									
20 400								0.045				
22 200							0.050		0.045			
24 100								0.060	0.050			
25 900								0.068	0.060			
26 800									0.065			0.003
27 800	0.073								0.070			0.008
29 200									0.075			0.009
29 600							0.075	0.085		0.005	0.005	
30 500									0.083			0.010
31 500								0.095			0.015	
31 800									0.093			0.018
32 700									0.100			0.023
33 700									0.110			0.030
33 900								0.110			0.018	
34 200	0.090											
35 100	0.095						0.103			0.020		
35 700								0.140			0.050	
37 000	0.100						0.125			0.070		
38 000	0.100									Failed.		
38 900			0.113				0.150			0.095		Failed.
39 800											Failed.	
40 700	0.110											
41 600			0.120									
41 800	0.113											
42 500	0.116		0.125									
43 500	0.118		0.126									

TABLE 52. (*Continued*).

38 000	0.505	0.458	0.403	0.003								
38 900	0.515	0.473	0.415	0.003								
39 800	0.525	0.483	0.420	0.005								
40 700	0.535	0.485	0.426	0.008	0.001							
41 600		0.503	0.428	0.015	0.001	0.000						
42 500		0.508		0.018	0.005	Failed.						
43 500				0.018	0.005							
44 400				Failed.	0.008							
46 300					0.009							
47 200					Failed.							
					0.010							
Order of testing	4	3	2				1	6	5			

NOTES. Nickel-steel strut No. 1 Camber at no load, 0.20 in. up.
" " " " 18 500 lb. per sq. in., 0.00 in.
" " " failure, 1.50 in. down.
" " 2 No record.
" " 3 Camber at failure, 2.25 in. down.

Carbon-steel strut No. 1 Camber at failure, 2.25 in. down.
Compressometer was dislodged twice during the test by the bending upward of the strut : the readings, therefore, are omitted from the table.
" 2 Camber at no load, 0.34 in. up.
" " 13 900 lb. per sq. in., 0.37 in. up
" " failure, 2.60 in. up.
" " 3 Camber at no load, 0.14 in. up.
" " 22 200 lb. per sq. in., 0.18 in. up
" " failure, 0.61 in. down.

TABLE 52. Compression Tests of Struts, 30 ft. Long: Structural Nickel Steel and Medium Carbon Steel.

Made on 2 160 000-lb. Hydraulic Testing Machine, Phœnix Iron Co.

Load, in pounds per square inch.	Nickel Steel.						Carbon Steel.					
	Temporary shortening, in inches.			Permanent set, in inches.			Temporary shortening, in inches.			Permanent set, in inches.		
	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3.	No. 1.	No. 2.	No. 3
11 200			0.120	0.000	0.000	0.000	0.095	0.122	0.117	0.000	0.000	0.000
12 000	0.130											
13 000		0.135					0.118	0.147	0.144		0.001	
13 900	0.170						0.128	0.152			0.001	
14 800	0.185						0.158	0.172	0.174		0.003	
15 700	0.195						0.178	0.184			0.003	
16 600							0.193	0.199	0.204		0.005	0.004
17 600							0.200	0.214			0.009	
18 500	0.240	0.235	0.220					0.224	0.234		0.010	0.009
19 400			0.230					0.237			0.010	
20 400			0.245				0.230	0.239	0.264		0.013	0.015
21 300			0.248				0.243		0.279			0.018
22 230			0.270				0.245	0.264	0.284		0.015	0.020
23 100	0.298	0.283	0.283				0.260		0.307			0.025
24 100		0.298	0.283				0.270	0.289	0.329		0.020	0.030
25 000		0.298	0.298				0.283		0.347			0.035
25 900		0.313	0.315				0.283	0.314	0.359		0.023	0.040
26 800			0.318				0.293		0.389			0.050
27 800	0.358	0.343	0.328				0.303	0.349	0.414		0.033	0.060
28 700			0.335						0.437			0.070
29 600	0.400	0.363	0.340	0.001				0.384		Failed.	0.041	Failed.
30 500			0.345									
31 500	0.430	0.388	0.345	0.001				0.399			0.053	
32 400		0.395	0.365								Failed.	
33 300	0.435	0.415	0.373	0.001								
34 200	0.445	0.433	0.375	0.001								
35 100	0.455	0.430	0.383	0.001								
36 100	0.470	0.433	0.398	0.001								
37 000	0.490	0.448	0.400	0.001								

TABLE 53.—TENSILE TESTS OF 6 BY 1-IN., 8 BY 2-IN., AND 16 BY 2-IN. EYE-BARS: NICKEL-STEEL EYE-BAR MATERIAL.

Size, in inches.	Head A—Pin-Hole.				Head B—Pin-Hole.				Length, in Feet, Back to Back of Pin-Holes.		Gauge length, in feet.	Original Dimensions. Area, in square inches.	Dimensions Fractured.		Load, in Pounds per Square Inch.		Elastic ratio, per cent.	Percentage of Elongation in:			Reduction of area, per cent.	Fracture	
	Original Dimensions.		Elongated, Diameter, in inches.	Excess, per cent.	Original Dimensions.		Elongated, Diameter, in inches.	Excess, per cent.	Before fracture.	After fracture.			Section, in inches.	Area, in square inches.	Yield point.	Ultimate strength.		12 inches.	10 feet.	20 feet.		Character.	Head A, Location, in feet
	Width, in inches.	Diameter, in inches.			Width, in inches.	Diameter, in inches.																	
6 by 1.	14.52	5.54	6.26	56.7	14.50	5.56	6.30	53.0	22.76	25.11	20.0	5.96	4.78 by 0.74	3.54	58 700	99 000	59.3	24.2	12.5	11.0	40.6	S. Aug.	3.4
6 " 1.	14.50	5.52	6.20	53.7	14.67	5.55	6.14	56.0	22.78	24.81	20.0	5.96	5.01 " 0.74	3.71	55 400	101 500	54.6	20.5	11.3	9.5	37.8	S. Aug.	4.8
6 " 1.	14.59	5.56	6.06	60.6	14.52	5.56	6.30	54.9	22.73	24.78	20.0	5.96	5.15 " 0.69	3.55	55 400	103 200	53.7	21.2	10.7	9.6	40.1	S. $\frac{1}{2}$ Cup.	4.6
6 " 1.	14.51	5.54	6.14	55.6	14.50	5.54	6.06	57.0	22.77	24.79	20.0	5.88	4.80 " 0.66	3.17	56 100	100 300	55.9	25.8	16.6	9.5	46.1	S. $\frac{1}{2}$ Cup.	3.1
																				19 ft.			
8 " 2.	19.66	7.02	7.80	60.8	19.72	7.02	7.84	61.5	22.29	24.45	19.0	16.04	6.69 " 1.51	10.10	48 800*	88 900	54.3	27.2	12.0	10.5	37.0	S. Irreg.	21.1
8 " 2.	19.56	7.02	8.00	57.5	19.72	7.02	7.70	61.1	22.34	24.66	19.0	16.24	6.88 " 1.62	11.14	52 300	90 800	57.6	23.8	12.0	11.3	31.4	S. Cup.	3.2
8 " 2.	19.68	7.02	7.82	50.6	19.59	7.02	7.82	58.6	22.22	24.76	19.0	16.08	6.50 " 1.44	9.36	54 400	90 800	59.9	26.8	14.3	12.5	41.8	S. Cup.	18.6
																				11 ft.			
16 " 2.	37.34	14.02	16.10	51.1	37.38	14.02	16.50	50.0	20.45	22.81	14.0	32.40	14.30 " 1.76	25.17	49 400	96 900	51.0	25.4	14.5	13.6	22.3	Cryst.	10.9

* 15% fine crystalline.

TABLE 54.—Tensile Tests of 6 by 1-in. Eye-Bars, up to the Yield Point. Eye-Bar Material—Nickel Steel—Stretch and Permanent Set.

Load on bar, in pounds per square inch.	*B* 1, Area = 5.96 in.		*B* 2, Area = 5.96 in.		*B* 3, Area = 5.96 in.		Load on bar, in pounds per square inch.	*B* 4, Area = 5.88 in.	
	Stretch.	Set.	Stretch.	Set.	Stretch.	Set.		Stretch.	Set.
.....	0.000	0.000	0.000	0.000	0.000	0.000	.....	0.009	0.090
8 400	0.047	0.000	0.063	0.000	.....	0.000	8 500	.....	0.000
16 800	0.100	0.000	0.144	0.000	.....	0.000	17 000	0.094	0.009
25 200	.....	.....	0.175	0.000	.....	.....	.....	.....	.....
33 600	0.222	0.000	0.231	0.000	0.231	0.000	34 000	0.206	0.000
41 900	.....	.....	0.300	0.006	.....	.....	.....	.....	.....
43 600	.....	.....	0.303	0.000	.....	.....	.....	.....	.....
45 300	0.300	0.002	0.325	0.000	0.313	0.000	45 900	0.288	0.006
46 100	.....	.....	0.328	0.006	.....	.....	.....	.....	.....
47 000	0.313	0.003	0.338	0.006	0.325	0.006	47 600	0.303	0.009
47 800	0.313	0.005	0.350	0.013	0.331	0.009	48 500	0.313	0.011
48 700	0.322	0.006	0.363	0.019	0.338	0.016	49 300	0.319	0.013
49 500	0.325	0.008	0.363	0.022	0.350	0.025	50 200	0.325	0.013
50 300	0.338	0.009	0.369	0.023	0.363	0.028	51 000	0.338	0.014
51 200	0.341	0.009	0.375	0.023	0.363	0.031	51 900	0.338	0.016
52 000	0.347	0.009	0.394	0.025	0.375	0.034	52 700	0.350	0.019
52 900	0.356	0.009	0.400	0.027	0.375	0.038	53 600	0.363	0.020
53 700	0.363	0.009	0.406	0.028	0.388	0.041	54 400	0.369	.....
54 500	0.369	0.009	0.406	0.031	0.394	0.044	55 300	0.381	0.023
55 400	0.375	0.009	0.419	0.034	..... (0.419)	0.050	56 100	0.394	0.027
56 200	0.388	0.011	0.428	0.041	0.444	0.056	57 000	0.406	0.038
57 100	0.394	0.013	.....	.....	.....	.....	.....	peeling	.....
57 900	0.406	0.016	.....	.....	.....	.....	.....	.....	.....
58 700	0.413 peeling	0.019	0.450	0.056	0.450	0.081	59 500	.....	0.338
59 600	0.431	0.038	.....	.....	.....	.....	60 400	.....	.....
60 400	.....	0.063	.....	.....	.....	.....	.....	.....	.....
67 100	....	.....	.....	.....	.....	0.506	.....	.....	....
Ultimate strength	99 000		101 530		103 200			100 300	
Order of testing	4		1		2			3	

The stretch and set were measured in a length of 15 ft.; they are expressed in decimals of an inch.

The first readings for Bars *B* 3 and *B* 4 were discarded because the pointer of the extensometer did not assume a permanent zero until after a load of 100 000 lb. had been applied.

TABLE 55.—TENSILE TESTS OF 8 BY 2-IN. AND 16 BY 2-IN. EYE-BARS, UP TO THE YIELD POINT. EYE-BAR MATERIAL—NICKEL STEEL—STRETCH AND PERMANENT SET.

Load on bar, in pounds per square inch.	8 BY 2-IN. BAR, *B* 1, AREA = 16.04 IN.		Load on bar, in pounds per square inch.	8 BY 2 IN. BAR, *B* 2, AREA = 16.24 IN.		Load on bar, in pounds per square inch.	8 BY 2-IN. BAR, *B* 3, AREA = 16.08 IN.		Load on bar, in pounds per square inch.	16 BY 2-IN. BAR *B* 4, AREA = 32.49 IN.	
	Stretch.	Set.		Stretch.	Set.		Stretch.	Set.		Stretch.	Set.
.....	0.000	0.000	.....	0.000	0.000	.....	0.000	0.000	.....	0.009	0.000
7 800	.....	.....	7 700	.....	.....	7 800	.....	.....	12 500	.....	.....
29 600	0.184	0.000	29 200	0.188	0.000	29 500	0.181	0.000	30 500	.....	.....
7 800	0.047	0.000	7 700	0.044	0.000	7 800	0.044	0.000	12 300	.....	.....
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.041	0.000	12 300	0.050	0.000
29 600	0.197	0.000	29 200	0.185	0.000	29 500	0.175	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.044	0.000	12 300	0.050	0.000
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
7 800	0.044	0.000	7 700	0.044	0.000	7 800	0.038	0.000	12 300	0.050	0.000
29 600	0.191	0.000	29 200	0.181	0.000	29 500	0.188	0.000	30 900	0.133	0.000
43 600	0.297	0.006	43 100	0.282	0.006	43 500	0.288	0.000	34 000	0.141	0.000
.....	.....	.....	44 600	0.360	0.009	.....	.....	.....	37 000	0.162	0.003
46 800	0.335	0.016	46 200	0.313	0.013	46 600	0.313	0.000	40 100	0.181	0.006
48 300	0.360 peeling	0.028	47 700	.....	0.016	48 200	0.319	0.000	43 200	0.206	0.011
49 900	0.385	0.041	49 300	.....	0.019	49 800	0.331	0.003	44 800	0.219	0.016
51 400	0.422	0.247	50 800	0.357	0.022	51 300	0.350	0.006	46 300	.....	0.019
.....	.....	.....	52 300	0.369 peeling	0.025	52 900	0.363	0.009	47 800	0.238	0.028
.....	.....	.....	53 900	0.394	0.038	54 400	0.288	0.013	49 400	0.256 peeling	0.031
.....	.....	.....	55 400	0.410	0.056	56 000	0.400 peeling	0.022	52 500	0.288	0.050
.....	.....	.....	57 000	0.457	0.088	57 500	0.419	0.034	55 600	0.344	0.094
.....	.....	.....	.....	.....	.....	59 100	0.588	0.200	.....	.....	.....
Ultimate strength	88 900			90 800			90 800			96 900	
Order of testing...	1			3			2			4	

The stretch and set were measured in a length of 15 ft. for *B* 1, *B* 2 and *B* 3, and 10 ft. for *B* 4; they are expressed in decimals of an inch.

The pointer of the extensometer did not assume permanent zero upon the removal of the load until after a load of 475 000 lb. had been put on the 8 in. bars; the first readings, therefore, were discarded.

The first three readings for the 16 in. bar were discarded, for the same reason

TABLE 56. – Comparison of Tensile Tests on Specimens and Full-Sized Bars.

Eye-Bar Material—Nickel Steel.

Cut from bar.	Dimension.	Load in Pounds per Square Inch.		Elastic ratio. Per cent.	Percentage of Elongation in:				Reduction of area. Per cent.	Remarks.
		Yield point.	Ultimate strength.		8 in.	12 in.	10 ft.	20 ft.		
Specimen Tests.										
6 by 1 in..	1-in. round.	62 300	104 300	59.7	18.8	....	....	....	53.3	Original.
6 " 1 "..	1 " "	56 400	102 800	54.9	18.8	....	....	....	47.0	Annealed.
6 " 1 "..	$1\frac{1}{4}$ by 1 in.	59 800	102 700	58.2	19.6	....	....	....	50.4	Original.
6 " 1 "..	$1\frac{1}{4}$ " 1 "	54 000	99 000	54.5	19.2	....	....	....	48.3	Annealed.
8 " 2 "..	$\frac{3}{4}$ " 2 "	58 100	98 600	58.9	21.8	....	....	....	50.9	Original. Bars *B*1 and *B*2. Yield point of No. 11 omitted
8 " 2 "..	$\frac{3}{4}$ " 2 "	52 800	95 700	55.2	20.7	....	....	....	47.8	Annealed. " " "
8 " 2 "..	$\frac{3}{4}$ " 2 "	55 200	99 500	55.5	19.8	....	....	....	47.6	Original. Bar *B*3.
8 " 2 "..	$\frac{3}{4}$ " 2 "	51 900	94 200	55.0	21.4	....	....	....	49.8	Annealed. " "
16 " 2 "..	$\frac{3}{4}$ " 2 "	59 100	103 600	57.0	20.8	....	....	....	47.2	Original.
16 " 2 "..	$\frac{3}{4}$ " 2 "	55 600	103 200	53.9	19.4	....	....	....	44.3	Annealed.
Full-Sized Tests.										
Mark.										
B 1.....	6 by 1 in.	58 700	99 000	59.3	....	24.2	12.5	11.0	40.6	
B 2.....	6 " 1 "	55 400	101 500	54.6	....	20.3	11.3	9.5	37.8	
B 3.....	6 " 1 "	55 400	103 200	53.7	....	21.2	10.7	9.6	40.4	
B 4.....	6 " 1 "	56 100	100 300	55.9	....	25.8	10.6	9.5	46.1	
B 1.....	8 " 2 "	48 300	88 900	54.3	....	27.2	12.0	19 ft. / 10.5	37.0	15 per cent. crystalline.
B 2.....	8 " 2 "	52 300	90 800	57.6	....	23.8	12.0	11.3	31.4	
B 3.....	8 " 2 "	54 400	90 800	59.9	....	26.8	14.2	12.5	41.8	
B 4.....	16 " 2 "	49 400	96 900	51.0	....	25.4	14.5	14 ft. / 13.6	22.3	Fine crystalline.

TABLE 57.—Tensile Tests on A. A. S. M. Specimens. Eye-Bar 16 by 2-in.

6 and 8-in. bars—Heat No. 17 749;

Cut from:	Original. Section, in inches.	Original. Area, in square inches.	Fractured section, in inches.	Load, in Pounds per Square Inch. Yield point.	Load, in Pounds per Square Inch. Ultimate strength.	Elastic ratio. Per cent.
6 by 1-in.	Diameter = 0.959	0.722	Diameter = 0.655	62 350	104 300	59.7
"	Diameter = 0.962	0.727	Diameter = 0.700	56 400	102 800	54.9
"	1.255 by 0.985	1.236	0.900 by 0.665	.....	102 400	.....
"	1.250 " 0.995	1.244	0.930 " 0.695	.....	98 600	.....
"	1.265 " 0.985	1.246	0.925 " 0.675	62 600	101 300	61.8
"	1.255 " 0.995	1.249	0.930 " 0.680	55 200	98 700	55.9
"	1.250 " 0.985	1.231	0.905 " 0.685	56 900	104 500	54.5
"	1.245 " 0.990	1.233	0.925 " 0.700	52 700	99 800	52.8
8 by 2-in.	0.760 " 1.985	1.509	0.505 " 1.465	*61 600	98 700	62.4
"	0.780 " 1.990	1.552	0.515 " 1.490	51 500	94 500	54.5
"	0.755 " 1.980	1.495	0.475 " 1.480	54 700	93 800	58.3
"	0.755 " 1.975	1.491	0.510 " 1.480	51 000	92 600	55.1
"	0.750 " 1.990	1.493	0.500 " 1.470	*60 900	99 900	61.0
"	0.765 " 1.985	1.518	0.530 " 1.510	54 000	96 800	55.8
"	0.755 " 1.985	1.499	0.515 " 1.495	58 700	101 800	57.7
"	0.755 " 1.990	1.502	0.555 " 1.550	54 600	98 900	55.2
"	0.740 " 1.985	1.469	0.520 " 1.535	59 900	103 500	57.9
"	0.760 " 1.985	1.509	0.510 " 1.510	53 000	96 100	55.2
"	0.765 " 1.980	1.515	0.525 " 1.460	50 400	95 400	52.8
"	0.755 " 1.980	1.495	0.500 " 1.465	50 700	92 300	54.9
16 by 2-in.	0.760 " 2.030	1.543	0.525 " 1.570	59 600	104 000	57.3
"	0.760 " 2.025	1.539	0.550 " 1.575	55 900	103 700	53.9
"	0.755 " 2.025	1.529	0.515 " 1.550	58 500	103 100	56.7
"	0.760 " 2.025	1.539	0.540 " 1.550	55 200	102 700	53.8

Pieces cut out from 6 by 1-in. Bar, *B*1, were turned to 1 in. in diameter and used for " " " " 6 by 1-in. Bars *B*2, 3 and 4, and piece *TSLEB*32, were tested on " " " " 8 by 2-in. and 16 by 2-in. bars, except *TSLEB*32, were tested on

* Autographic curves are abnormal at yield point.

† Broke 1½ in. from end gauge mark

MATERIAL—NICKEL STEEL—CUT FROM 6 BY 1-IN., 8 BY 2-IN., AND EYE-BARS.

16-in. bars—Heat No. 17 673.

PERCENTAGE OF ELONGATION IN:				Reduction of area.	Character of fracture.	SPEED OF BREAKING.		Location of specimen.	Remarks.
2 in.	4 in.	6 in.	8 in.			At yield point.	At break.		
37.0	26.5	21.7	18.8	53.3	S ½ Cup.	Very slow.	1 in. in 2 min.	All	Original.
35.0	25.5	21.0	18.8	47.0	S ⅛ Cup.	" "	"	half	Annealed.
40.5	26.5	21.3	19.0	51.6	S Cup.	1 in. in 10 min.	1 in. in 1 min.	way	Original.
37.5	25.5	21.2	18.5	48.1	S ¾ Cup.	"	"	between	Annealed.
41.0	28.3	23.0	20.6	49.9	S Ang.	"	"	edge	Original.
38.5	27.0	22.3	19.5	49.4	S Cup.	"	"	and	Annealed.
40.5	26.5	21.7	19.3	49.6	S Ang.	"	"	middle	Original.
40.0	28.3	22.7	19.5	47.4	S ¾ Cup.	"	"	of bar.	Annealed.
40.0	30.0	23.3	20.3	51.0	S Ang.	1 in. in 20 min.	1 in. in 2 min.	Middle.	Original.
40.0	28.5	23.3	+20.0	50.6	S Cup.	"	"	"	Annealed.
35.0	31.0	25.7	22.4	53.0	S Irreg.	"	"	Edge.	Original.
45.0	31.5	25.3	21.8	49.4	S Ang.	"	"	"	Annealed.
44.0	31.2	25.7	22.4	50.8	S Ang.	"	"	Middle.	Original.
42.0	29.0	23.3	20.3	47.3	S Ang.	"	"	"	Annealed.
42.0	30.5	25.0	22.0	48.6	S Cup.	"	"	Edge.	Original.
37.0	28.5	23.7	20.8	43.7	S Cup.	"	"	"	Annealed.
39.0	26.0	20.0	+17.5	45.7	S Ang.	"	"	Middle.	Original.
40.0	29.5	24.0	20.8	49.0	S Ang.	"	"	"	Annealed.
45.5	32.0	25.8	22.0	49.5	S Cup.	1 in. in 10 min.	1 in. in 1 min.	Edge.	Original.
45.0	31.0	25.3	22.0	51.0	S Ang.	1 in. in 20 min.	1 in. in 2 min.	"	Annealed.
42.0	29.5	24.3	20.5	46.6	S Cup.	"	"	Middle.	Original.
34.0	24.5	22.0	19.3	43.0	S Cup.	"	"	"	Annealed.
42.0	29.5	24.0	21.0	47.8	S Ang.	"	"	Edge.	Original.
37.0	27.5	22.3	19.5	45.6	S Ang.	"	"	"	Annealed.

Coefficient of Elasticity determination.
150 000-lb. Riehle machine at Riehle Bros. Co.
200 000-lb. Olsen machine at Drexel Institute.

TABLE 58.—Bending Tests on Plain Specimens Cut from 6 by 1-in., 8 by 2-in., and 16 by 2-in. Eye-Bars.

Eye-Bar Material—Nickel Steel.

Tests Made on Hydraulic Bending Machine, Pencoyd, Pa.

Cut from eye-bar:	Heat treatment.	Angle of bend, in degrees.	2 × (Radius of Bend): Inches.	2 × (Radius of Bend): "t"	Opening at 1 in. from inside edge, in inches.	Remarks.
6 by 1-in.	Original.	191	15/8	0.9	1 3/16	No cracks.
"	Annealed.	190	15/16	0.9	1 3/16	Skin broken.
"	Original.	192	15/16	0.9	1 3/16	Crack 1 in. long, another 3/4 in. long.
"	Annealed.	191	1	1.0	1 1/4	Skin broken.
"	Original.	191	1 1/16	1.1	1 3/8	Three small cracks.
"	Annealed.	188	13/16	0.8	1	No cracks.
"	Original.	194	1 1/4	1.3	1 1/2	One small crack.
"	Annealed.	190	15/16	0.9	1 3/16	No cracks.
8 by 2-in.	Original.	175	2	1.0	2	"
"	Annealed.	174	2 1/8	1.1	2 1/4	"
"	Original.	173	2	1.0	2 1/8	"
"	Annealed.	171	1 3/4	0.9	1 7/8	"
"	Original.	175	2 1/4	1.1	2 1/4	"
"	Annealed.	173	1 5/8	0.8	1 11/16	One slight crack in corner only.
16 by 2-in.	Original	173	2 5/8	1.3	2 1/2	No cracks.
"	Annealed.	177	2 1/2	1.3	2 3/8	"

"t" = thickness of material.

6 and 8-in. bars were rolled from Heat No. 17 749; 16-in. bars from Heat No. 17 673.

APPENDIX B. (PART I.)

TABLE 59.—Chemical Analyses and Specimen Test Results of Material from Which Full-Sized Tests Met Specifications. *B*-1200—Blackwell's Island Bridge—Nickel-Steel Eye-Bars.

Heat No.	Chemical Analysis.					Unannealed.				Annealed.			
	C.	P.	S.	Mn.	Ni.	Elastic limit.	Ultimate strength.	Elongation.	Reduction.	Elastic limit.	Ultimate strength.	Elongation.	Reduction.
15 119	0.36	0.010	0.027	0.75	3.36	56 980	100 500	18.7	35.9	50 450	94 080	27.0	50.5
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
15 121	0.32	0.014	0.029	0.65	3.36	55 740	95 260	21.2	37.1	50 880	87 740	26.2	50.9
16 122	0.37	0.016	0.031	0.70	3.34	57 500	99 350	19.5	30.8	55 100	97 340	19.7	54.4
16 122	0.37	0.016	0.031	0.70	3.34	57 500	99 350	19.5	30.8	55 100	97 340	19.7	54.4
16 068	0.32	0.015	0.030	0.65	3.22	57 230	96 510	20.0	34.5	51 840	87 470	24.2	44.3
1 088	0.38	0.012	0.029	0.60	3.36	55 200	100 400	20.0	29.2	53 360	90 830	21.2	48.5
16 245	0.37	0.010	0.022	0.74	3.36	60 650	106 900	17.5	25.6	54 020	97 560	20.0	41.0
16 230	0.40	0.013	0.026	0.67	3.34	57 890	104 000	16.2	25.8	53 720	91 520	25.0	47.6
16 257	0.38	0.012	0.033	0.72	3.26	59 300	104 500	16.2	27.4	53 160	97 290	23.7	45.3
16 313	0.43	0.014	0.033	0.75	3.28	56 180	119 900	15.0	28.6	51 260	105 260	21.0	43.9
2 139	0.38	0.010	0.028	0.76	3.28	58 550	102 200	20.0	32.0	54 080	96 200	18.2	46.1
1 105	0.36	0.010	0.030	0.73	3.30	58 940	105 500	19.2	28.3	51 860	86 400	25.0	44.9
16 225	0.41	0.010	0.034	0.70	3.36	60 280	110 100	16.2	34.0	54 620	101 900	20.5	41.6
1 114	0.41	0.012	0.037	0.62	3.52	59 270	104 400	14.0	19.0	52 400	97 400	21.2	29.7
16 227	0.34	0.016	0.032	0.70	3.28	55 380	104 100	20.0	34.6	55 640	91 060	21.2	29.0
16 227	0.34	0.016	0.032	0.70	3.28	55 380	104 100	20.0	34.6	55 640	91 060	21.2	29.0
16 222	0.45	0.013	0.025	0.70	3.26	64 780	113 500	18.1	33.5	50 840	101 200	21.25	46.4
16 255	0.41	0.012	0.026	0.80	3.30	62 780	115 400	16.2	31.8	52 120	104 200	20.0	34.2
16 308	0.34	0.010	0.030	0.66	3.28	62 940	107 200	18.7	25.4	54 220	93 360	22.5	48.6
16 248	0.41	0.012	0.030	0.80	3.28	66 360	116 000	15.0	28.8	60 160	104 800	22.5	42.8
16 260	0.40	0.010	0.030	0.78	3.28	55 240	114 090	16.0	28.9	53 780	103 600	18.5	37.2
16 316	0.38	0.010	0.028	0.70	3.40	55 610	95 140	20.0	38.7	52 560	97 940	22.5	44.1
10 120	0.41	0.010	0.035	0.70	3.40	64 420	119 800	16.2	23.7	55 020	103 200	21.2	42.2
2 161	0.38	0.010	0.035	0.74	3.26	57 560	106 200	22.5	45.4	50 220	100 960	21.25	33.6
1 115	0.43	0.016	0.025	0.75	3.32	63 960	110 100	16.0	28.9	54 250	104 200	18.70	38.2
16 290	0.39	0.010	0.035	0.75	3.36	61 260	108 300	17.2	30.8	54 010	95 540	23.7	51.3
16 272	0.40	0.012	0.025	0.79	3.25	66 760	111 600	14.0	20.1	55 860	102 600	20.0	40.2
16 263	0.36	0.012	0.028	0.74	3.46	65 250	107 050	17.5	27.4	48 040	94 070	23.7	45.0
16 281	0.44	0.010	0.026	0.72	3.28	63 200	116 500	15.0	20.8	55 600	105 800	22.5	32.2
16 280	0.42	0.011	0.039	0.69	3.26	67 640	117 000	13.7	21.5	51 460	101 700	17.5	31.5
16 292	0.43	0.010	0.035	0.80	3.36	66 080	112 100	14.5	25.3	55 140	98 320	21.2	42.5
16 239	0.37	0.010	0.030	0.65	3.46	60 400	101 600	18.7	33.3	53 950	92 160	21.2	38.7
16 239	0.37	0.010	0.030	0.65	3.46	60 400	101 600	18.7	33.3	53 950	92 160	21.2	38.7
16 303	0.44	0.010	0.025	0.80	3.76	60 620	113 400	17.2	26.4	55 850	101 500	21.0	43.9
16 238	0.37	0.012	0.033	0.85	3.30	65 430	110 200	18.7	32.1	50 440	96 640	15.0	22.3
10 096	0.40	0.010	0.028	0.62	3.36	55 760	94 000	20.0	34.4	49 670	88 380	21.2	40.1
16 231	0.39	0.012	0.031	0.80	3.30	63 900	110 100	16.0	28.9	55 400	104 300	20.0	30.0
2 145	0.36	0.015	0.030	0.75	3.28	56 140	105 400	17.0	35.2	55 360	97 700	24.2	50.5
16 226	0.37	0.012	0.030	0.78	3.26	57 840	109 100	18.7	32.4	55 630	96 740	23.7	47.2
16 285	0.45	0.011	0.030	0.78	3.26	67 120	112 400	15.0	27.4	52 080	103 400	20.0	37.4
16 236	0.36	0.012	0.035	0.65	3.28	61 860	104 000	16.2	25.0	52 200	91 720	21.2	45.2
16 322	0.42	0.010	0.028	0.77	3.30	66 080	116 200	16.7	31.0	58 060	102 100	20.0	30.9
16 223	0.45	0.011	0.030	0.75	3.28	68 300	118 300	15.0	24.2	54 250	110 800	21.0	38.8
16 224	0.38	0.011	0.033	0.72	3.36	63 440	105 400	15.0	17.3	52 130	95 620	18.25	20.3
10 115	0.40	0.012	0.032	0.87	3.26	64 480	105 200	14.5	18.8	55 980	105 900	20.0	35.3
1 110	0.38	0.010	0.034	0.72	3.46	60 000	102 200	18.7	28.2	51 540	94 800	21.2	41.0
1 123	0.43	0.916	0.030	0.72	3.32	62 260	110 300	17.2	30.9	62 140	102 100	21.2	41.0
16 258	0.37	0.010	0.026	0.73	3.28	57 420	104 000	18.7	32.9	53 000	94 200	23.7	40.6
10 111	0.40	0.010	0.030	0.64	3.34	62 360	110 200	17.06	32.2	56 050	98 100	21.2	37.6
2 165	0.37	0.013	0.037	0.80	3.28	62 600	113 200	17.5	15.9	51 600	99 480	20.0	46.9
10 116	0.37	0.014	0.029	0.70	3.36	61 570	100 300	20.0	31.8	54 270	84 100	25.0	49.1
16 311	0.37	0.010	0.033	0.75	3.36	62 450	105 800	18.7	34.4	54 880	97 620	21.2	45.5
16 256	0.40	0.012	0.022	0.73	3.28	67 700	112 200	14.5	24.1	53 250	101 600	22.5	47.3
16 851	0.38	0.010	0.028	0.64	3.48	56 230	100 500	20.0	32.7	53 980	95 400	26.2	53.0
16 861	0.39	0.010	0.030	0.69	3.34	55 900	103 800	18.7	30.7	52 900	94 440	24.7	48.5
14 566	0.41	0.011	0.031	0.66	3.60	60 900	105 900	19.7	38.3	56 560	97 020	23.5	40.1

TABLE 59.—(*Continued*).

Heat No.	Chemical Analysis.					Unannealed.				Annealed.			
	C.	P.	S.	Mn.	Ni.	Elastic limit.	Ultimate strength.	Elongation.	Reduction.	Elastic limit.	Ultimate strength.	Elongation.	Reduction.
14 571	0.44	0.012	0.029	0.78	3.30	65 950	113 100	16.2	30.8	61 680	105 200	21.2	42.4
14 543	0.37	0.013	0.025	0.80	3.30	64 280	107 500	18.7	36.1	52 320	102 900	20.0	33.5
16 859	0.38	0.010	0.035	0.66	3.34	55 500	99 180	20.0	35.0	58 340	90 220	24.7	13.5
14 559	0.47	0.013	0.023	0.72	3.40	59 350	115 700	13.2	28.1	48 220	108 000	16.2	28.9
16 268	0.42	0.010	0.022	0.78	3.36	55 620	117 000	14.5	13.8	56 200	105 400	18.7	39.1
16 270	0.42	0.010	0.036	0.75	3.36	62 400	115 200	16.2	30.0	51 000	101 000	21.5	45.6
14 545	0.40	0.010	0.036	0.66	3.26	55 680	104 100	21.2	36.9	50 800	95 060	22.5	45.1
16 305	0.44	0.010	0.025	0.73	3.38	60 740	116 700	17.5	29.5	52 040	103 300	22.0	41.3
14 555	0.46	0.010	0.036	0.65	3.36	57 300	116 400	15.0	22.4	62 620	108 200	20.5	38.2
14 530	0.40	0.010	0.029	0.63	3.30	55 220	101 400	23.2	43.7	50 100	91 560	23.2	54.5
14 558	0.39	0.022	0.031	0.85	3.40	65 750	110 400	17.0	31.1	58 820	101 400	22.2	50.3
12 738	0.41	0.010	0.034	0.72	3.54	62 020	102 900	18.7	32.8	52 960	95 300	22.5	50.5
14 552	0.44	0.016	0.031	0.63	3.42	63 120	113 500	12.5	18.6	56 720	90 500	23.7	48.7
1 116	0.45	0.015	0.028	0.80	3.40	66 320	117 500	13.7	18.8	59 900	109 700	21.2	34.2
16 279	0.44	0.010	0.031	0.82	3.26	55 440	112 500	16.2	29.6	51 840	102 700	20.5	44.9
12 041	0.39	0.012	0.030	0.75	3.62	61 160	107 050	18.2	29.5	56 400	94 900	25.0	44.5
12 059	0.43	0.013	0.028	0.70	3.64	59 940	112 500	12.5	23.0	55 380	101 400	18.7	25.6
12 054	0.38	0.017	0.029	0.63	3.50	65 630	101 600	16.2	23.1	50 140	86 700	22.0	39.8
12 081	0.40	0.012	0.029	0.69	3.40	60 800	107 400	18.7	26.7	56 730	96 320	23.7	41.6
12 064	0.43	0.016	0.032	0.78	3.58	61 700	102 300	19.5	24.1	58 060	96 800	21.2	43.7
16 795	0.40	0.010	0.030	0.65	3.58	62 220	103 000	18.7	26.3	56 260	95 200	23.7	46.7
12 055	0.38	0.010	0.037	0.69	3.56	62 320	102 800	18.5	28.5	56 360	88 140	25.0	49.5
12 049	0.38	0.010	0.030	0.78	3.59	64 170	103 800	17.0	27.7	57 160	89 950	25.0	52.7
16 306	0.42	0.010	0.030	0.78	3.32	60 480	109 800	17.5	33.6	50 100	98 960	21.2	44.3
16 320	0.42	0.010	0.027	0.75	3.32	62 600	110 700	16.2	32.0	53 720	95 740	21.2	42.8
12 037	0.39	0.012	0.030	0.69	3.46	61 200	105 000	19.7	28.5	61 510	93 010	28.0	50.8
16 262	0.42	0.010	0.022	0.73	3.39	61 300	116 500	15.0	31.3	54 450	104 200	21.2	40.4
16 328	0.36	0.010	0.031	0.76	3.32	64 490	108 850	17.0	19.4	58 220	95 460	21.7	38.9
16 330	0.38	0.014	0.038	0.70	3.32	60 950	106 900	18.7	31.4	57 200	96 190	21.2	44.1
14 548	0.41	0.010	0.036	0.73	3.40	61 800	108 650	20.0	38.7	59 180	102 350	22.5	46.2
16 791	0.39	0.010	0.030	0.69	3.52	62 800	104 600	20.0	32.2	55 400	93 300	25.0	48.7
12 014	0.39	0.012	0.032	0.66	3.42	55 760	102 900	20.0	28.5	53 200	92 500	21.2	46.3
12 080	0.40	0.010	0.025	0.75	3.50	59 090	107 400	20.0	22.5	55 190	96 970	24.5	40.9
14 743	0.42	0.010	0.035	0.71	3.54	59 930	111 800	16.2	27.4	56 380	100 600	23.2	47.1
14 742	0.42	0.013	0.032	0.65	3.46	56 500	104 900	18.7	29.3	54 080	96 240	25.7	45.1
16 309	0.37	0.010	0.032	0.79	3.26	57 160	107 700	20.0	38.5	51 260	97 820	26.2	49.8
14 747	0.38	0.012	0.030	0.69	3.56	60 140	100 400	20.7	37.1	57 060	92 560	27.5	49.5
12 737	0.41	0.012	0.034	0.65	3.26	57 600	104 500	18.7	31.4	56 940	93 480	22.2	47.0
12 776	0.40	0.010	0.029	0.66	3.62	59 880	102 900	18.7	29.6	52 980	94 460	22.5	45.1
12 707	0.41	0.010	0.029	0.68	3.30	62 640	106 700	15.0	31.2	58 740	97 200	23.7	49.3
1 444	0.38	0.012	0.034	0.63	3.30	55 340	98 700	20.0	29.7	52 280	86 720	23.7	49.6
12 261	0.42	0.016	0.027	0.70	3.50	61 900	103 500	21.2	33.5	52 940	96 560	25.0	42.9
12 013	0.41	0.012	0.032	0.68	3.40	58 860	103 500	17.5	27.3	48 250	88 240	25.0	45.3
12 003	0.37	0.010	0.034	0.66	6.64	56 760	102 700	20.0	28.0	53 960	95 720	23.7	40.7
12 884	0.45	0.010	0.027	0.65	3.60	55 580	109 300	16.2	26.2	59 060	100 500	24.7	41.6
12 193	0.39	0.012	0.029	0.70	3.48	60 600	100 400	22.0	38.2	51 810	92 040	23.7	45.6
12 740	0.38	0.013	0.032	0.69	3.34	61 020	105 400	17.5	34.0	52 800	93 460	22.5	50.7
12 062	0.37	0.011	0.032	0.75	3.46	59 720	101 300	20.5	40.9	52 640	89 160	23.7	47.7
12 340	0.40	0.012	0.027	0.62	3.36	62 840	109 400	15.0	25.8	53 940	94 120	23.2	42.0
12 709	0.41	0.010	0.029	0.68	3.36	61 000	103 700	18.7	29.8	48 330	96 540	25.0	46.1
12 340	0.40	0.012	0.027	0.62	3.36	62 840	109 400	15.0	25.8	53 940	94 120	23.2	42.0
11 434	0.40	0.012	0.037	0.65	3.64	58 340	100 300	19.5	31.3	53 700	89 920	24.2	41.9
12 359	0.39	0.010	0.032	0.74	3.50	64 920	104 700	20.5	33.7	55 600	99 500	23.7	39.7
12 358	0.36	0.010	0.030	0.60	3.50	59 420	97 460	21.2	34.1	57 400	91 600	22.0	38.1
12 346	0.40	0.010	0.027	0.80	3.62	65 860	104 500	18.0	34.0	59 170	94 060	27.0	49.1
12 343	0.37	0.010	0.027	0.69	3.64	64 420	108 800	17.5	34.3	54 060	95 100	23.2	42.8
12 452	0.38	0.010	0.028	0.76	3.44	62 950	106 700	18.7	25.1	57 800	97 680	23.2	45.4
12 727	0.37	0.010	0.032	0.58	3.44	57 320	95 340	20.0	35.5	48 030	88 820	23.7	48.3
12 725	0.39	0.010	0.025	0.68	3.64	60 140	102 000	17.5	37.5	57 500	96 100	23.7	42.8
12 773	0.40	0.010	0.030	0.60	3.34	58 700	102 600	18.7	33.0	55 420	90 700	26.0	50.0
12 728	0.36	0.011	0.027	0.60	3.36	59 860	103 300	21.2	35.4	57 480	93 320	25.0	49.1
12 539	0.39	0.011	0.031	0.77	3.40	59 940	100 800	20.0	25.7	51 500	91 960	25.5	44.1
12 450	0.36	0.010	0.030	0.54	3.36	59 940	99 450	16.2	30.1	54 410	91 800	23.7	43.7
11 437	0.37	0.010	0.029	0.59	3.53	59 660	101 900	18.2	26.4	59 950	95 920	21.7	34.8
13 429	0.36	0.010	0.033	0.63	3.50	61 040	99 360	18.7	30.4	55 370	93 080	24.5	45.9
14 553	0.43	0.018	0.032	0.73	3.34	63 740	112 800	14.7	26.8	57 720	103 800	18.7	28.9
12 345	0.43	0.012	0.033	0.69	3.50	61 340	108 400	16.7	30.1	56 106	98 760	22.0	39.2

APPENDIX B. (PART II.)

TABLE 60.—Result of Tests That Meet Specifications. Blackwell's Island Bridge—Nickel-Steel Eye-Bars.

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft.	Reduction of area. Percentage.	Character of fracture.
15 119	16 × 2	3.36	54 260	87 710	6.80	12.3	Crystalline. Broke in head.
15 121	16 × 1 7/8	3.36	52 460	86 270	10.00	36.1	Silky.
15 121	16 × 1 15/16	3.36	50 590	85 841	11.10	40.5	95% silky.
15 121	16 × 1 7/8	3.36	51 780	88 280	10.32	28.6	Cup. 65% silky.
16 122	16 × 1 7/8	3.34	51 320	86 360	6.55	8.7	Crystalline. Broke in head.
16 122	16 × 1 7/8	3.34	52 746	90 080	7.00	11.8	Crystalline. Broke in head.
16 068	16 × 1 7/8	3.22	51 150	89 840	10.60	10.3	60% crystalline and 40% silky. Broke in head.
1 088	16 × 1 7/8	3.36	49 850	82 540	14.10	33.8	Silky; irregular.
16 245	16 × 1 7/8	3.36	49 810	87 970	14.77	39.6	" Cup.
16 230	16 × 2	3.34	51 280	90 120	9.22	....	Fine granular. Broke in head.
16 257	16 × 2	3.26	50 660	89 470	13.21	44.5	Silky cup.
16 313	16 × 2 1/8	3.28	50 072	98 980	10.55	38.9	Irreg. silky. Trace crystalline.
2 139	16 × 2 1/16	3.28	52 990	88 310	18.44	38.8	Silky. 90% cup.
1 105	16 × 1 7/8	3.64	48 051	85 535	13.05	35.3	" angular.
16 225	16 × 2 1/16	3.36	48 102	88 137	13.50	21.4	Irreg. silky and fine granular.
1 114	16 × 2	3.52	48 296	85 446	16.61	26.9	Silky 90% cup.
16 227	16 × 2	3.29	51 314	86 357	14.88	4.3	" 70% "
16 227	16 × 2	3.28	49 340	88 623	11.55	38.9	" angular.
15 222	16 × 2	3.26	51 020	95 392	12.11	40.4	" "
16 255	16 × 1 3/4	3.30	48 170	98 840	10.00	33.9	Fine granular.
10 308	16 × 1 15/16	3.28	49 315	82 410	17.61	44.7	Silky ½ cup.
16 248	16 × 1 7/8	3.28	49 150	91 160	16.16	29.1	" angular.
16 260	16 × 1 7/8	3.28	49 210	95 380	12.27	37.2	" "
16 316	16 × 1 15/16	3.40	48 150	82 950	12.11	39.7	" irregular.
10 120	16 × 1 7/8	3.40	48 400	84 500	12.83	35.9	" "
2 161	16 × 1 3/4	3.26	51 240	83 400	10.83	48.8	" cup.
1 115	16 × 2 1/16	3.32	49 100	98 350	11.22	36.7	" square. Trace granular.
16 290	16 × 1 7/8	3.36	49 360	92 080	16.00	44.0	" cup.
16 272	16 × 2 1/8	3.28	51 070	98 000	18.38	8.7	Broke in head.
16 263	16 × 2	3.46	53 000	87 660	15.94	44.2	Silky ½ cup.
16 281	16 × 2	3.28	49 290	94 890	10.55	28.3	Irreg. 80% crystalline. bal. silky.
16 280	16 × 2	3.26	49 040	96 700	12.16	27.4	Square. Near silky.
16 292	16 × 2	3.36	52 360	95 880	13.90	38.4	Silky cup.
16 239	16 × 1 7/8	3.26	48 130	84 160	12.66	21.8	75% crystalline, bal. irreg. silky.
16 239	16 × 1 7/8	3.26	48 150	89 020	14.00	36.5	Silky irregular.
16 303	16 × 1 7/8	3.26	48 110	86 000	9.66	28.7	75% crystalline, bal. silky.
16 238	16 × 1 3/4	3.30	50 090	91 000	15.94	41.3	Silky cup.
10 096	16 × 2	3.36	49 220	90 020	11.77	34.2	" irregular.
16 231	16 × 2 1/8	3.30	48 230	95 600	10.83	40.5	" "
2 145	16 × 2 1/16	3.28	53 110	90 670	13.77	6.8	" 25% angular, bal. square.
16 226	16 × 2	3.26	48 920	93 320	11.11	....	Too long to pull to fracture.
16 285	16 × 2 1/16	3.26	39 960	90 480	13.00	44.5	Irreg. 60% silky, bal. crystalline.
16 236	16 × 1 7/8	3.28	49 330	85 250	14.80	44.5	Silky angular.
16 322	16 × 1 15/16	3.30	48 270	93 500	9.66	41.4	" irregular.
16 223	16 × 2 1/16	3.28	48 250	91 010	13.33	41.9	Irregular silky.
16 224	16 × 2	3.36	49 850	89 260	12.16	....	Too long to pull to fracture.
10 115	16 × 2	3.26	49 200	95 760	13.16	32.7	Irregular silky.
1 110	16 × 2	3.46	49 090	90 390	17.11	34.2	Irreg. 50% silky, bal. granular.
1 123	16 × 1 7/8	3.32	50 530	91 810	11.05	17.0	Square granular.
16 258	16 × 1 7/8	3.28	48 260	83 630	11.50	31.8	Irregular silky.
10 111	16 × 1 7/8	3.34	51 070	85 490	9.44	....	Granular. Broke in head.
2 165	16 × 1 7/8	3.28	50 140	83 300	14.27	43.0	Silky irregular.
10 116	16 × 1 7/8	3.36	48 120	80 480	10.94	26.6	" angular.
16 311	16 × 1 7/8	3.36	50 260	85 980	9.77	38.8	" "

TABLE 60. (*Continued.*)

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft	Reduction of area. Percentage.	Character of fracture.
16 256	16 × 2	3.28	54 040	98 060	11.94	31.9	Silky irregular.
16 851	16 × 1 7/8	3.48	48 290	83 520	12.16	34.2	" "
16 861	16 × 1 7/8	3.34	48 060	86 180	9.11	44.5	" "
14 566	16 × 2 1/16	3.60	57 050	87 770	8.16	....	Granular. Broke in head.
13 571	16 × 1 7/8	3.30	48 520	90 030	10.55	43.4	Irregular. 40% silky, bal. gran.
14 543	16 × 2	3.30	48 160	87 290	10.94	45.4	Irregular. 60% silky, bal. gran.
16 859	16 × 1 15/16	3.34	48 890	82 740	11.22	44.5	Silky angular.
14 559	16 × 2	3.40	48 120	90 780	9.77	37.2	20% silky, bal. crystalline.
16 268	16 × 2	3.36	52 970	87 520	6.05	....	50% silky, bal. crystalline. Broke in head.
16 270	16 × 1 15/16	3.36	48 050	90 920	10.33	44.4	Silky irregular.
14 545	16 × 1 13/16	3.26	48 110	84 970	9.16	32.2	Irregular. 30% silky, bal. crystalline.
16 305	16 × 2	3.38	53 200	88 030	6.00	....	Granular. Broke in head.
14 555	16 × 1 7/8	3.36	52 870	95 010	7.55	....	Granular. Broke in head.
14 530	16 × 1 15/16	3.30	51 340	86 050	7.33	....	30% silky, bal. crystalline. Broke in head.
14 558	16 × 1 7/8	3.40	48 200	91 920	13.66	38.8	Silky angular.
12 738	14 × 1 11/16	3.54	50 400	85 470	13.05	43.8	Silky ½ cup.
14 552	14 × 1 11/16	3.42	48 270	91 760	10.33	37.9	Silky irregular.
1 116	16 × 2 1/16	3.40	48 240	93 900	9.72	32.8	20% silky, bal. cryst.
16 279	16 × 2 1/16	3.36	56 380	94 430	12.05	33.5	Silky irregular.
12 041	16 × 1 13/16	3.62	58 000	89 860	14.27	32.7	" angular.
12 059	16 × 1 13/16	3.64	48 980	88 870	9.44	34.9	" irregular.
12 054	16 × 1 15/16	3.50	50 080	86 170	10.55	41.6	70% silky, bal. cryst.
12 081	16 × 1 13/16	3.40	51 320	87 980	7.22	....	30% " " " " Broke in head.
12 064	16 × 1 13/16	3.58	52 160	92 150	10.00	33.3	Silky irregular.
16 795	16 × 2 1/8	3.58	48 290	86 130	10.94	41.9	" cup.
12 055	16 × 1 15/16	3.56	48 140	87 400	10.88	41.1	" irregular.
12 049	16 × 1 3/4	3.54	52 210	87 620	11.60	43.9	" ½ cup.
16 306	16 × 2 1/8	3.32	48 230	82 650	8.44	27.4	" square.
16 320	16 × 2 1/8	3.36	48 310	84 470	9.88	35.5	" irregular.
12 037	16 × 2 1/8	3.46	52 490	89 560	6.44	....	Granular. Broke in head.
16 262	16 × 2 1/16	3.34	48 240	87 680	13.88	43.5	80% silky, bal. cryst.
16 328	16 × 2 1/8	3.32	48 210	82 730	9.11	38.2	Silky irregular.
16 330	16 × 2 1/8	3.32	48 170	81 670	12.44	45.8	" ½ cup.
14 548	14 × 1 7/8	3.40	53 330	91 890	8.83	....	Crystalline. Broke in head.
16 791	16 × 2 1/8	3.52	49 230	82 250	9.16	46.3	Silky irregular.
12 014	16 × 2	3.42	48 040	85 580	10.27	38.9	" "
12 080	16 × 2	3.50	48 010	87 280	10.11	39.3	50% silky and bal. cryst.
14 743	16 × 1 7/8	3.54	55 900	98 060	10.77	34.2	10% " " "
14 742	16 × 2 1/16	3.46	51 350	90 180	9.38	....	Fine crystalline. Broke in head.
16 309	16 × 2 1/8	3.26	55 930	92 800	7.28	....	Crystalline. Broke in head.
14 747	16 × 1 3/4	3.56	54 112	87 810	11.05	42.0	Silky irregular.
12 737	16 × 2	3.46	55 690	86 320	12.33	33.9	" angular.
12 776	16 × 2	3.62	54 820	90 220	7.22	....	Granular. Broke in head.
12 707	16 × 2 1/16	3.30	56 270	89 130	6.50	....	Crystalline. " " "
1 444	16 × 2	3.30	52 860	83 020	11.27	28.2	Angular. ½ silky and bal. crystalline.
12 261	16 × 2 1/8	3.50	50 000	85 500	12.55	42.9	Silky irregular.
12 013	16 × 2 1/8	3.40	52 050	86 170	6.88	....	Granular. Broke in head.
12 003	16 × 2 1/8	3.64	54 390	88 970	10.38	40.2	Silky irregular.
12 884	16 × 2 1/8	3.44	49 160	80 360	10.00	37.2	" angular.
12 193	16 × 1 15/16	3.48	48 240	81 330	11.16	34.4	" irregular.
12 740	16 × 2	3.34	48 460	87 190	12.77	42.0	" angular.
12 062	16 × 1 15/16	3.54	60 570	87 790	11.00	....	Granular. Broke in head.
12 340	14 × 1 15/16	3.62	48 260	87 560	12.05	45.3	Silky angular.
12 709	16 × 2	3.36	51 130	87 670	8.50	....	80% crystalline. Broke in head.
12 340	16 × 2 1/8	3.36	52 080	89 286	7.66	....	Granular. Broke in head.
11 434	16 × 1 15/16	3.64	51 170	87 420	6.83	....	" " " "

TABLE 60.—(*Continued.*)

Heat No.	Section.	Percentage of nickel.	Elastic limit, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Percentage of elongation in 18 ft.	Reduction of area. Percentage.	Character of fracture.
12 359	16 × 1 15/16	3.50	57 260	96 930	6.00	....	Granular. Broke in head.
12 358	14 × 1 15/16	3.50	48 140	83 550	11.66	35.5	Irregular. 60% cryst. and bal. silky.
12 346	14 × 1 7/8	3.62	58 210	97 920	12.16	42.7	Silky ½ cup.
12 343	16 × 1 3/4	3.64	57 500	94 260	6.00	....	Granular. Broke in head.
12 452	14 × 1 3/4	3.44	56 000	95 090	9.06	32.0	Silky irregular.
12 727	16 × 2 1/16	3.44	52 440	95 420	6.55	....	75% silky, bal. gran. Broke in head.
12 725	16 × 2 1/16	3.64	50 320	85 910	9.78	44.7	Silky irregular.
12 773	16 × 2 1/16	3.34	49 030	83 840	10.94	46.4	" cup.
12 728	16 × 2 1/16	3.36	56 680	85 800	14.28	....	" irregular. Broke in head.
12 539	14 × 1 15/16	3.40	52 270	89 620	9.00	37.8	" cup.
12 450	14 × 1 15/16	3.36	52 920	86 680	9.11	4.1	" angular.
11 437	16 × 2 1/8	3.53	53 030	89 070	6.05	....	Granular. Broke in head.
12 429	16 × 2	3.50	50 060	89 740	12.83	33.4	Silky square.
14 553	14 × 1 7/8	3.34	53 090	87 560	6.55	....	Crystalline. Broke in head.
12 345	16 × 2	3.50	51,020	86 420	14.77	41.6	50% silky and 50% fine crystalline.

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PAPERS AND DISCUSSIONS

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FORESTS AND RESERVOIRS IN THEIR RELATION TO STREAM FLOW WITH PARTICULAR REFERENCE TO NAVIGABLE RIVERS.

By H. M. Chittenden,* M. Am. Soc. C. E.

To be Presented November 4th, 1908.

The following paper is presented at this time with the purpose of eliciting from the Society membership the results of observation and experience touching the important matters of which it treats. They are vital features of one of the chief living questions before the public to-day, and an expression of views by men accustomed to look at things from a practical standpoint cannot fail to be of great value to our legislators upon whom the ultimate responsibility for action must rest.

While the author's views traverse, to some extent, currently accepted theories, they are based upon long observation and study and are what seem to be unavoidable conclusions therefrom; but he is committed to no theory, as such, and his mind is entirely open to conviction upon any point in which his opinions may be shown to be errone-

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Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and when finally closed, the paper, with discussion in full, will be published in *Transactions*.

ous. His sympathies are wholly on the side of the present movement for the conservation of our natural resources, and, so far as this paper takes issue with certain tendencies of that movement, it is only for the purpose of inquiring whether such tendencies are not really inimical to the cause to which they pertain.

With this preliminary statement, the author will take up the first part of his paper, viz., the influence of forests upon stream flow.

Forests and Stream Flow.

The commonly accepted opinion is that forests have a beneficial influence on stream flow:

(1) By storing the waters from rain and melting snow in the bed of humus that develops under forest cover, preventing their rapid rush to the streams and paying them out gradually afterward, thus acting as true reservoirs in equalizing the run-off.

(2) By retarding the snow-melting in the spring and prolonging the run-off from that source.

(3) By increasing precipitation.

(4) By preventing erosion of the soil on steep slopes and thereby protecting watercourses, canals, reservoirs and similar works from accumulations of silt.

There are many subsidiary influences, but, broadly stated, the above propositions cover the ground. They were first given general currency nearly forty years ago through the writings of Sir Gustav Wex, Chief Engineer on the Improvement of the Danube, whose treatise was translated into English by the late General Weitzel, of the Corps of Engineers. Wex's theories were stoutly resisted at the time by many European engineers and still find only a limited acceptance in the profession;* though in the popular mind they have gained ground and in the United States are now accepted practically without question.

To establish by definite proof the truth or falsity of these propositions is an extremely difficult task. One would not think so, indeed, to judge from the cheerful confidence with which the popular thought accepts them; but it is nevertheless so. The elements of the problem are so many and conflicting, the necessary evidence is so hard to get, and comparative records are of such recent date, that precise demon-

*Almost simultaneously with the publication of Wex's treatise, a similar work was published in France by M. F. Vallée taking exactly the opposite view of the question.

stration is scarcely possible. The popular belief is based upon a fact and an assumption forming together a basis for a conclusion. The fact is that forests in the eastern portion of the United States have disappeared to a large extent within the past century. The assumption is that floods and low waters in the same region are more frequent and severe than before the forests were cleared away. The conclusion is that these assumed conditions must be due to the disappearance of the forests. *Post hoc, ergo propter hoc* is the argumentative process relied upon, and little effort is made to consider whether there may not be some other and more satisfactory explanation. The author will attempt to analyze the problem from a theoretical standpoint and will then cite existing records so far as these are sufficiently long-continued to be worth anything. He will consider, first, the effect of the forests where stream flow results from rain alone, and, next, where it results in part from melting snow.

Effect of Forests upon the Run-Off from Rainfall.—The first of the above propositions—the retentive action of the forest bed—may be accepted at once as strictly true for average conditions. It is not true for extreme conditions—great floods and excessive low waters—the conditions that determine the character and cost of river control. Consider an inclined-plane surface, practically impervious to water, with a layer of sand covering some small portion of it, and let a uniform spray of water be applied to the entire surface. Assume that the temperature and rate of evaporation are relatively low. As soon as the spray begins, water commences to flow from the uncovered surface, but not for a time from that covered by the sand. After a while it begins to trickle from the sand, increasing in volume until the sand is thoroughly saturated, after which it flows off in as great quantity per unit area as from the uncovered portion. If the spray is stopped, the water immediately ceases to flow from the uncovered area, but continues in diminishing quantity from the covered area until it finally ceases altogether; but not all the water that fell on this area has run away. The sand has retained some portion of it and given it off in evaporation, so that the total run-off per unit area is somewhat less than on the uncovered portion. If the shower be long-continued and the rate of evaporation very low, the difference of total run-off per unit area from the two surfaces will be very slight.

Suppose now that the temperature and rate of evaporation are high

and that the spray works intermittently. If the showers are small in volume and the intervals between them long, the sand may retain nearly or quite all of the individual showers and give them off in evaporation, so that there will be no run-off whatever.

Between these two extreme conditions, the covered area will exert a greater or smaller regulative effect upon the run-off. The retentive power of the sand will be less as the slope of the surface upon which it rests increases, or it will be greatest when the surface is nearly horizontal and least when it is nearly vertical.*

Now in Nature this ideal illustration is never fully exemplified in the cleared land and the forest. There is nearly everywhere a marked retentive capacity in the bare soil. In newly plowed ground it is probably greater than in the forest. Moreover, certain crops, like heavy grass or grain, obstruct the flow of water almost as much as the forest cover. On the other hand, the furrows of cultivated fields, drainage ditches, roads and, particularly, the pavements and roofs of towns, greatly accelerate the run-off; so that, while the full contrast of the ideal example does not exist in Nature, the principle of the illustration applies perfectly. That is, there are times when the percentage of retention in the forest bed is 0, and there are other times when it is 100; or, there are times when so much water comes that the forest bed can hold none of it and there are times when so little comes that it holds it all. Between these extremes there are periods when it holds more or less and gives up less or more and exercises a

*Since the above was written the author has noticed, in the report of the hearing on House Resolution 208 before the Committee on the Judiciary, that Gifford Pinchot, Assoc. Am. Soc. C. E., Chief of the Forest Service, used an illustration very similar to that given above, except that he failed to carry it to its logical conclusion. Addressing the Committee, February 27th, 1908, he said: "I have in my hand here a photograph of a denuded hillside. After the forest has been removed rain falls on that hillside and runs off rapidly, as the water I drop upon the photograph does now, and disappears instantly (illustrating). If, on the other hand, I place a forest cover on the hillside, that is exactly analogous in texture and effect with this piece of blotting paper, and drop the water slowly upon it, we would find that, instead of running off slowly at the bottom, the water is held (illustrating with blotting paper). Part of it runs off, but, as soon as the absorbent quality of the paper or the forest floor has time to take effect, the water is kept and drips gradually for a considerable length of time off the hill into the stream. This is an exact illustration of the way in which the forest controls the stream flow on that hillside."

Mr. Pinchot should have completed his illustration. He should have continued to sprinkle the paper long enough and heavily enough to have saturated the paper completely in order to show that the water would then flow from the paper as rapidly as from the uncovered area; and he should then have explained that this condition represents what always happens in the forest in times of great flood. Then he should have sprinkled the paper intermittently in small quantities, and at such long intervals that the warm air of the room would evaporate all of the absorbed water, and that none whatever would flow away. He should then have explained that this condition represents what always takes place in the forest in times of great drought.

corresponding influence upon the run-off. There is another important condition not exemplified in the illustration, and that is that the forest areas are scattered everywhere, the ground has an infinite variety of slope, the showers never fall uniformly over an entire water-shed, and the final result in the total run-off is the summation of thousands of tributary results.

It is true, therefore, as popularly understood, that, in periods of ordinary rainfall with sufficient intervals for the forest bed to dry out somewhat, forests do exert a regulative effect upon run-off. They modify freshets and torrents and prolong the run-off after storms have passed, and thus realize in greater or less perfection the commonly accepted theory.

This result utterly fails, however, in those periods of long-continued, widespread and heavy precipitation, which alone cause great floods in the large rivers. At such times the forest bed becomes completely saturated, its storage capacity exhausted, and it has no more power to restrain floods than the open country itself. Moreover, the fact that the forest bed has retained a portion of earlier rainfall and is yielding it up later to the streams, produces a condition that may be worse than it would be in a country cleared of forests. Really great floods in large rivers are always, as is well known, the result of combinations from the various tributaries. It is when the floods from these tributaries arrive simultaneously at a common point that calamitous results follow. Any cause which facilitates such combinations is, therefore, a source of danger. Now, unquestionably, in a heavily wooded water-shed, forests do have a tendency in this direction. When a period of heavy storms occurs, spreading over a great area, continually increasing in intensity, the forests, by retaining some portion of the earlier showers and paying them out afterward, do produce a general high condition of the river which may greatly aggravate a sudden flood arising later from some portion of the water-shed. That the forest does promote tributary combinations, there would seem to be no question, and that it may therefore aggravate flood conditions necessarily follows. It is not contended that this increase is ever very great, but it is contended that forests never diminish great floods and that they probably do increase them somewhat. The forests are virtually automatic reservoirs, not subject to intelligent control, and act just as the system of reservoirs once proposed by the French Government

for the control of floods in the River Rhone would have acted, if built. These reservoirs were to have open outlets, not capable of being closed, which were intended to restrain only a portion of the flow. A careful study of their operation in certain recorded floods showed that they would actually have produced combinations more dangerous than would have occurred without them.

Consider now periods of extreme drought and grant that, as a general rule, springs and little streams dry up more completely than when forests covered the country, although this difference is very greatly exaggerated in the popular mind.* At first thought one would conclude that, since the springs and streams make up the rivers, these also ought now to show a smaller low-water flow than formerly. This, however, is not the case. The difference between the former low-water flow of a spring or rivulet and what it is now is relatively an insignificant quantity. Most of such water sources yield but a small fraction of a cubic foot per second. Whether these small quantities are a trifle more or less cuts very little figure in the aggregate; and so it counts but little in the flow of a great river whether some of its extreme sources lose a portion of a volume that is already inappreciable. When the summer showers come, however, there is a marked difference. At such times the forests not only hold the water back—they often swallow it completely. Small showers that make a perceptible run-off in the open are often practically all absorbed in the leaves of the trees. Heavier showers, that make freshets in the open, are largely absorbed in the leaves and forest bed and pass off in evaporation; so that, contrary to the general view, the evaporation from the forest is greater at such times than in the open country and the run-off from summer precipitation is less. A single shower may produce a sufficiently greater run-off in a deforested area to more than offset the diminished low-water flow for several weeks.† Now on most of the smaller streams quantity of flow is a more important matter than

*The term "as a general rule" is used, for it is by no means absolute. In particular the drainage of low swamp lands leads off into the streams, in dry weather, waters that formerly remained or passed off in evaporation, and in such cases even the low-water flow is greater than it used to be. In 1895 the author saw an example of this on the Scioto River near the outlet of the great Scioto swamp which had recently been drained. A small mill was able to operate during the low-water season more regularly than formerly. Tile drainage, now so widely used, has the same tendency.

†So far as the author is aware, Colonel T. P. Roberts, of Pittsburg, Pa., was the first to call attention to this characteristic of stream flow.

natural uniformity of flow, particularly in the summer time. The day of the small mill, which was so dependent upon such uniformity, is past. The modern water power invariably seeks uniformity by artificial regulation, and the ups and downs of its sources of supply are abolished in its storage. Therefore it does not matter nearly as much that the run-off of the small streams be uniform as that it yield a good flow of water; and if forests diminish the total low-water supply, this fact more than offsets the gain in uniformity. Likewise the great rivers swallow up and equalize the small irregularities of their head-waters and actually experience a somewhat larger low-water flow than if their water-sheds were still thickly forested. Thus, while forests may decrease somewhat the extreme range between maximum and minimum run-off on very small water-sheds, they do not do so on great ones, which are combinations of very small ones. At the same time it seems certain that forests decrease somewhat the total run-off from water-sheds small or great.*

Influence of Forests upon Snow-Melting.—The second proposition —that forests have a beneficial effect upon the run-off from snow-melting—is quite as firmly fixed in the popular belief as that just considered, but has even less foundation in fact. It is a relation that can be definitely traced, and it can be demonstrated that the effect of forests upon the run-off from snow is invariably to increase its intensity. This results from two causes, one affecting the falling of the snow and the other its melting.

In the first place, forests break the wind, prevent the formation of drifts, and distribute the snow in an even blanket over the ground. In the open country, the snow is largely heaped into drifts, their size depending upon the configuration of the ground, the presence of wind breaks, and the prevalence and force of the wind. These drifts form admirable reservoirs and in the high mountains are the most perfect known. Forests prevent their formation entirely.

The period of snow-melting begins in the open country much earlier than in the forests. At first the melting is due mainly to the direct

*This subject was ably discussed by Mr. Raphael Zon, of the Forest Service, Department of Agriculture, in *Transactions*, Am. Soc. C. E., Vol. LIX, pp. 494-495. He states among other things that "the quantity of water available for stream flow from forested water-sheds, all other conditions being equal, is less than from non-forested water-sheds"; that "the forest soil receives least precipitation, next comes meadow land, and lastly tilled land"; that "in the forest, only the upper layer of the soil is moister than in the open, the lower layers being always drier." This discussion is well worth perusal.

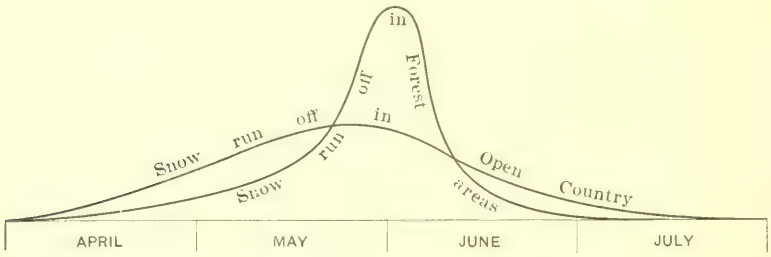
action of the sun's rays before there is sufficient warmth in the general atmosphere to produce any effect. The thinly covered areas melt off first and the streams experience a diurnal rise and fall following the warmth of day and the frost of night. Nothing like a flood ever arises from such melting.

Under forest cover this action is interfered with more or less, depending upon the density of the shade. Even after the ground in the open is entirely bare, except under the drifts, the forest areas may still be covered with an unbroken layer of snow. It is generally, though erroneously, considered that this delay is beneficial, by carrying farther into the summer the release of the winter precipitation and giving it more time to soak into the ground; but in fact this benefit does not result. The water from the first melting of the snow blanket does not sink into the ground but into itself. Snow is like a sponge. A panful will shrink to one-fourth of its volume, or less, before any free water appears. The author has seen an 8-ft. covering of snow dwindle to 2 ft., with the ground beneath it still comparatively dry.

The forest shade thus holds the snow, which gradually becomes saturated from its own melting, until the heat and warm rains of late spring or early summer arrive, the soft air everywhere pervading the forest depths and finding a maximum exposure of surface to the melting influences. A cubic yard of snow which in a great drift might stand 27 ft. deep with a square foot of exposure, may here lie with a depth of 1 ft. and 27 sq. ft. of exposure. The result is that when the final melting begins the whole body of snow disappears very rapidly, rushing from every direction into the streams, swelling them to their limit and often causing disastrous freshets. The active melting lasts but a short time, and there is little opportunity for the water to soak into the ground. The delay in melting, caused by the forest shade, has simply operated to concentrate it into a shorter period and increase the intensity of the resulting freshet. It comes so fast that the greater portion of it cannot be utilized at the time and is lost altogether unless intercepted by reservoirs.

In the open country, on the other hand, the drifts last for weeks after the snow has entirely disappeared from the forest, and continue to yield a supply of water far into the summer. The period of active melting in the open may have lasted four months, that in the forest scarcely as many weeks. Fig. 1 illustrates, in a general way, the

processes just described. These curves apply more particularly to the forests of the Rocky Mountains, where the author has had exceptional opportunities for studying their action. In the northwest corner of Wyoming and contiguous portions of the adjoining States, lies an elevated region of probably 20 000 sq. miles, which is the source of nearly all the great river systems of the West. It is a very remarkable region in this respect. Its average altitude is about 7 500 ft., and it is in large part covered with a dense evergreen forest. At the very summit of this elevated region is that singular section now visited annually by thousands of tourists—the Yellowstone Park. The opening of the tourist season in spring occurs just about the time of active snow-melting, and the most onerous and difficult task of those in charge of the road system of the Park is to get the roads into condi-



GRAPHIC ILLUSTRATION OF SNOW MELTING IN FORESTS AND OPEN COUNTRY, ROCKY MOUNTAIN REGION.

Fig. 1.

tion for the first travel. This frequently has to be done while the snow still lies deep on the ground. It was the repeated execution of this task that first drew the author's attention to the fact that, as a general rule, the floods of this region are forest floods, and that the same conditions of precipitation which force the forest streams out of their banks produce only moderate effects in the open. The traditional "June rise" comes mainly from the mountain forests.

The photographs presented herewith were taken about the middle of June in a year of heavy snowfall and only two days before the tourist season opened. Plate CVII, Fig. 1, shows an east and west road through a dense forest of lodge-pole pine at an altitude of 8 200 ft. It shows very effectively the deep, even blanket of snow everywhere covering the ground, except along a narrow strip at the roots of the

FIG. 1.—SNOW IN FOREST. YELLOWSTONE PARK, JUNE 13TH, 1899. ALTITUDE, 8 200 FT.

FIG. 2.—SNOW-DRIFT, YELLOWSTONE PARK, JUNE 13TH, 1899. ALTITUDE, 7 800 FT.

trees on the north side of the road, where the sun had access through the opening in the tree-tops caused by the 30-ft. clearing for the roadway. Plate CVII, Fig. 2, taken practically at the same time, shows one of the great drifts in the open country, which it was impossible to avoid in locating the road.

At this time a period of very warm weather had set in with frequent rains. Severe floods followed which did great injury to the roads and bridges, not only in the mountains, but for a considerable distance below. Within two weeks the snow had practically disappeared in the forests, but in the open country the drifts, like that in the photograph, continued until the middle of July, giving forth a continuous supply of water.

A most illuminating article, and one which every one interested in the subject should read, was published in *Science* for April 10th, 1896. It gives the results of observations in the mountains of Nevada for over 25 years, during which "extensive tracts of timber" were cut off "to the very ground," and new growths had been well started. It was found that springs, which were active after the land was cleared, dried up when the new forest growth developed; "that the water supply from the mountains is greater and more permanent now than it was before the timber was cut off"; that freshets were no more "frequent or violent than before the trees were cut off," and that "spring floods were less frequent." The greatly increased loss due to evaporation in the forest was pointed out. This results partly from the vast extent of surface on the ground exposed to the air and partly from exposure on the leaves and branches of the trees.

"The foliage on this class of trees being as heavy in winter as in summer, the branches catch an immense amount of the falling snow and hold it up in mid-air for both sun and air to work upon; and only those who have had experience of the absorbing power of the dry mountain air can form any idea of the loss from that source." Moreover "the trees absorb from the soil quite as much water as would be evaporated by the action of the sun in the absence of the shade."

The writer states that

"the strongest force at work to save our rivers is the drifting winds which heap up the snow in great banks; and in this the trees are a constant obstacle." He declares that "close observers, after long years of study, have been led to believe that if there is any difference in the

flow of streams and the size of springs before and after the trees are cut from above them, the balance is in the favor of the open country."*

In the current literature upon this subject one invariably encounters the same fallacious assumption, that because the forests delay melting their action is therefore beneficial. The fact is entirely overlooked that delay means concentration and greater intensity of run-off, while the open country prolongs the melting and gives a more even distribution. If the true action of forests in this respect, however, is rarely recognized by public writers, it is recognized, though perhaps unconsciously, by those who are benefitted by it. The monthly reports of the Weather Bureau in the Rocky Mountain region are instructive reading in this connection. The following are a few extracts from those sent in to the Central Office of the Western Montana district at Helena:

"Where there is no timber to break the force of the winds solid drifts of considerable depth have collected." * * * "The snowfall has been very light and the drifts are not large or solid enough to furnish an adequate flow of water in the streams." * * * "In some sections the winter's snowfall has been the lightest for many years, and as there is little likelihood that the later snows will form solid drifts, it is practically certain that the flow of water in most streams will be inadequate for irrigation and mining purposes."

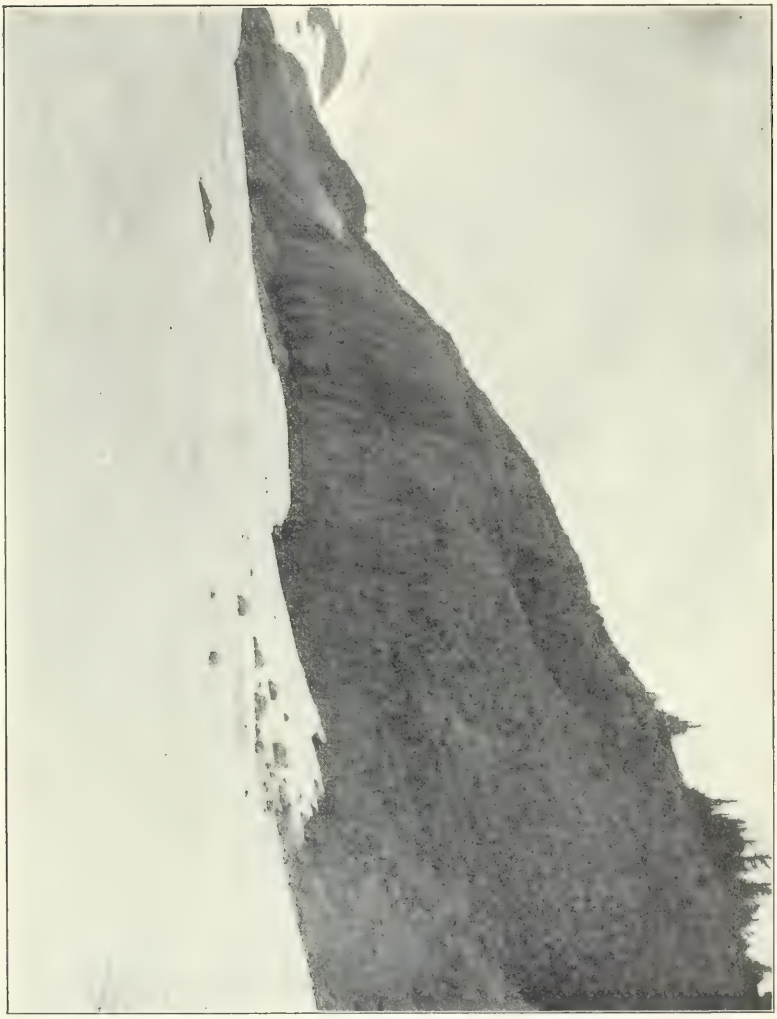
These extracts, which could be multiplied indefinitely, show how well the practical ranchman understands the value of snow-drifts. It has always been a mystery to the author that writers will persist in statements like the following which appears in one of the ablest addresses at the recent Conservation Conference in Washington:

"The possibility of irrigation depends largely on the preservation of the forest cover of the mountains, which catches and holds the melting snows, and thus forms the great storage reservoirs of nature."

The forests destroy the reservoirs and the flow would be more uniform, prolonged and plentiful, if they were not there.

In will doubtless be urged that while the foregoing conclusions may hold for an elevated and densely wooded region, they will not hold

*The author recalls only a single other writer who has set forth this matter in accordance with the facts, and that was an anonymous correspondent in a recent issue of the *Pacific Sportsman*. His view of the case is summarized in rather terse language as follows: "Trees in the mountains make floods in the spring." * * * "Snow in the timber melts too fast. The timber keeps it from drifting." * * * "The agency which maintains the river is the snow in the huge drifts." * * * "That (the drift) is your reservoir that feeds the living streams of summer time." * * * "The timber has nothing to do with the water supply but is a result of the water supply."



IN THE BASKET.

for a lower altitude, warmer climate and different kind of forest. In reply it may be said that in proportion as the conditions described prevail, they apply everywhere. In deciduous forests where the foliage is absent during seasons of snowfall and melting, the winds have greater play in winter and the sunlight in spring and there is, of course, less difference between the forests and the open country; but while the difference is less it is not obliterated altogether, and in hilly regions, like the Adirondacks and the White Mountains, it exists in full force. The author is very familiar with the region of Western New York—having been reared on a farm nearly on the divide between the waters of the Ohio and Lake Erie—a beautifully wooded country, deciduous growths prevailing, and one of the snowiest regions in the United States. While there is less drifting in the open and more in the woods than in high mountains, still it is strictly true that the open-country drifts outlast the forest snows just as the latter outlast the thin snows in the open.

A striking example of the action of forests on snow-melting may be seen in the mountains of the Pacific Coast. Here are the densest forests in the world, the deepest beds of humus, and the most perfect reservoir effect so long as it is in action. Yet in this very region, particularly around Puget Sound, are to be found some of the most torrential streams in the country. This fact is largely due to the distribution of snowfall caused by the forests. Conditions like the following are constantly developing. Heavy snow storms sweep over the forest-covered mountains. The snow cannot drift, for the dense woods break the wind. A great deal of it does not reach the ground at all, but hangs on the branches and undergrowth all the way from the highest tree tops down. This covering is often so dense as to prevent cruising operations altogether, because the cruisers cannot see the timber through the impenetrable screen of snow. Of an 18-in. fall, perhaps 12 in. is on the trees and the rest spread evenly on the ground. To show what now happens, let an illustration be drawn from the opposite process of drying clothes. When the housewife has finished her washing and wishes to dry the clothes, she does not set them out in a basket, where it would take weeks for them to dry, but spreads them upon the ground or hangs them on a line, so that the sun and air can reach them on all sides. So these forests increase, by a thousand-fold, the exposed area of the snow over what it would be if heaped in Nature's clothes

baskets (the great drifts), and give it the maximum possible exposure to the melting influences whenever these shall arrive. As a general rule these snow storms are followed by warm southerly winds and rains—the rains frequently heavy in themselves—and rain and snow join hands, two storms in one, and rush down to the ocean in tremendous freshets and floods. The Skagit River, the largest in Washington except the Columbia, and a very considerable stream, has been known to rise 1 ft. per hour for 16 hours, and this where the stream has a fall of 4 ft. to the mile and carries off its floods very rapidly. Plate CX, Fig. 1, taken on another stream with only 480 sq. miles of water-shed above it, shows the terrific power of these streams that come down from the most densely wooded and perfectly protected water-shed in existence. The great flood of 1906 in this section, was a perfect demonstration, not only of the vast intensifying effect of forests upon floods due to snow-melting, but of the utter helplessness of the forest bed, when saturated with long rains, to restrain floods.

The same effect was very manifest in the great flood of 1907 in the valley of the Sacramento River, California. The tributaries on the east side come down from the densely wooded slopes of the Sierras; those on the west side from the bare or sparsely wooded slopes of the Coast Range. If the forest theory be true, these smooth western slopes should send down a greater flow for the same precipitation than the eastern slope. Exactly the reverse seems to have been the case. For the period, March 17th-26th, the precipitation on the Puta Creek water-shed, on the west side (805 sq. miles), averaged 22.7 in. The maximum resulting run-off per second per square mile for one day was 39.1 cu. ft. Directly across the valley on the Sierra slope, the precipitation on the American River water-shed (2 000 sq. miles), averaged 14.6 in. for the same period, and the maximum daily discharge was 48.7 cu. ft. per sec. per sq. mile. Considering the fact that unit run-off for the same conditions is always less, the greater the water-shed, this result is quite remarkable. It is undoubtedly due to the action of the Sierra forests on snow-melting and again illustrates the inability of forests to exercise any restraining influence upon great floods.*

*In the paper, "The Flood of March, 1907, in the Sacramento and San Joaquin River Basins, California," by Messrs. Clapp, Murphy and Martin, published in *Proceedings*, Am. Soc. C. E. for February, 1908, the authors say, "In the Sierras the greater part of the precipitation is normally in the form of snow, and the magnitude of floods depends largely on the rate of melting. A heavy warm rain on deep, freshly fallen snow produces a maximum run-off."

ON THE LINE.

During the spring of 1908 occurred a record-breaking flood in Western Montana, nearly all the streams on both sides of the Continental Divide going far over their banks. As might have been predicted, this occurrence was promptly cited as another example of the effect that a forest-barren country has upon floods. Nevertheless it is as certain as anything of this kind can be, that if the country affected by this extraordinary downpour (in some places breaking all previous records) had been thickly forested, and the ground still covered, as it would have been, with a solid layer of saturated snow, the flood would have far exceeded in magnitude and destructiveness that which actually took place. Wherever forests existed in the higher altitudes they did have this effect.*

Having now considered the influence of forests upon stream flow from a theoretical standpoint, let the records themselves be examined as far as they are available. These records in the United States, unfortunately, are not so useful as might be wished, because of their brevity. No continuous records on any of our streams run back for more than eighty years, and most of them less than half as far. This is far short of the 200 years considered by certain European engineers who investigated Wex's theories as the minimum period "necessary in order to draw a reliable conclusion" upon this subject. It does indeed seem absurd to take present-day records, as is constantly done, and draw conclusions one way or the other as to comparisons with the past, of which records are entirely wanting; but such as they are, a few of these records are given in Table 1. They include in most cases both high and low water, although the low-water records cannot, in the nature of the case, be of very much value. Works of channel improvement on most of the streams have probably affected somewhat the low-water stages for the same discharge, while, as is well known, a given stage, even in a natural stream, does not mean the same discharge at different times.† It is really the discharge of the streams rather than the stage that forms the correct basis for comparison; but data for discharge are almost wholly wanting.

An examination of these records shows how utterly impossible it is to find anything in them to support the current theory of forest

*In the Weather Bureau report, Montana section for June, 1908, it is stated that "the rainfall was phenomenally heavy over most of this district, and, combined with the water from the rapidly melting snow in the high mountains, caused unprecedented floods in nearly all streams."

†During the past 20 years the low-water stage of the Mississippi at St. Paul has been materially modified by reservoir action.

TABLE 1.—Gauge Records of Certain Rivers of the United States.

Highest and Lowest Stages for Each Year.

Mississippi.

Year.	St. Paul.		St. Louis.	
	Highest.	Lowest.	Highest.	Lowest.
1785	....	....	40.6*	....
1826	....	....	? 33.7	....
1828	....	....	36.4	....
1838	....	....	27.0	....
1843	....	....	27.2	....
1844	....	....	41.3	....
1845	....	....	32.4	....
1846	....	....	25.0	....
1849	....	....	27.4	....
1851	....	....	36.5	....
1852	....	....	28.0	....
1853	....	....	30.0	....
1855	....	....	37.1	....
1856	....	....	27.4	....
1858	....	....	37.0	....
1860	....	....	....	0.6
1861	....	....	25.5	1.3
1862	....	....	31.5	3.5
1863	....	....	18.0	0.0
1864	....	....	20.3	1.2
1865	....	....	26.8	1.2
1866	....	2.2	26.8	5.6
1867	....	3?0	28.2	1.2
1868	....	2.0	24.1	1.0
1869	16.1	2.8	29.2	4.7
1870	....	....	26.2	5.2
1871	....	....	21.8	2.8
1872	? 7.7	1.9	23.0	2.4
1873	16.4	2.4	25.5	4.6
1874	11.6	3.0	18.4	2.8
1875	18.0	2.1	30.0	2.3
1876	11.0	1.9	31.8	5.0
1877	7.7	1.8	26.5	7.0
1878	6.7	0.6	25.7	5.6
1879	10.8	0.9	21.2	3.4
1880	15.3	1.7	25.5	2.7
1881	19.7	3.3	33.6	7.6
1882	13.3	2.9	32.0	2.8
1883	12.5	1.5	34.8	4.5
1884	10.3	1.8	28.1	3.0
1885	7.4	1.9	27.1	2.0
1886	8.2	1.2	27.0	1.4
1887	9.6	0.8	20.6	0.9
1888	14.4	2.4	29.4	3.2
1889	4.5	0.8	24.6	2.5
1890	7.0	0.6	20.5	2.8
1891	6.4	0.1	23.7	3.0
1892	12.6	1.0	36.0	1.0
1893	14.7	0.6	31.6	— 0.2
1894	11.8	0.2	23.3	0.2
1895	4.6	0.2	23.4	— 0.5
1896	10.7	0.9	27.7	3.7
1897	18.0	2.3	30.9	— 0.5
1898	10.7	2.8	27.2	0.3
1899	11.0	2.4	25.7	— 1.0
1900	6.6	0.7	23.5	— 2.5
1901	7.5	1.2	22.5	— 2.0
1902	7.5	1.1	26.8	— 1.0
1903	13.5	2.5	38.0	0.6
1904	9.9	2.6	33.6	— 0.1
1905	14.8	2.0	30.2	— 0.3
1906	13.3	4.9	26.2	3.0
1907	13.6	1.3	....	....

* "L'année des grandes eaux."

TABLE 1.—(*Continued.*)

OHIO.

Year.	Pittsburg.		Cincinnati.		Louisville.	
	Highest.	Lowest.	Highest.	Lowest.	Highest.	Lowest.
1810	32.0	.....	....	....	Upper	Gauge
1813	29.0	.....	....	....	....	....
1816	33.0	.....	....	....	....	....
1832	34.0	.....	64.2	....	40.8	....
1840	26.9	.....	....	....	....	....
1846	25.0	.....	....	....	....	....
1847	26.0	.....	63.6	....	....	....
1848	23.0	.....	....	....	....	....
1851	30.9	.....	....	....	20.3	1.5
1852	31.9	.....	....	....	....	....
1853	31.9	.....	....	....	....	....
1855	18.0	2.1	....	....	....	....
1856	19.6	0.3	....	....	....	1.1
1857	21.4	0.0	....	....	....	....
1858	26.0	0.5	43.1	2.5	19.1	1.5
1859	22.0	1.1	55.5	3.3	33.8	2.0
1860	29.7	2.8	49.2	5.3	....	....
1861	30.9	1.1	49.4	5.1	....	....
1862	30.0	0.3	57.3	2.3	....	....
1863	16.0	0.1	42.7	2.5	....	....
1864	18.6	1.0	45.1	3.1	....	....
1865	31.4	1.4	56.2	5.7	....	....
1866	15.4	0.4	42.5	4.7	18.6	2.8
1867	22.6	0.0	55.7	3.0	37.6	1.8
1868	20.6	0.0	48.2	5.1	22.3	3.0
1869	19.6	0.7	48.7	5.3	24.3	2.8
1870	18.0	1.3	55.2	3.8	....	....
1871	19.0	1.2	40.5	2.7	....	....
1872	20.6	1.6	41.7	3.0	21.0	2.0
1873	25.6	1.6	44.4	3.7	18.3	2.3
1874	22.4	1.1	47.9	2.3	22.4	1.8
1875	25.0	0.4	55.3	4.2	30.3	2.7
1876	26.0	0.2	51.7	6.2	32.5	3.7
1877	25.0	0.7	53.7	3.2	29.9	2.2
1878	24.6	0.0	41.3	4.3	15.7	2.9
1879	20.0	0.1	42.7	2.5	19.6	2.2
1880	22.0	0.3	53.1	3.7	30.0	2.8
1881	28.0	0.6	50.6	1.9	22.5	1.7
1882	21.9	0.6	58.6	6.1	37.4	4.7
1883	27.6	0.1	66.3	3.6	43.8	3.2
1884	34.4	0.3	71.0	2.7	46.7	2.0
1885	23.0	1.0	46.0	2.5	22.2	2.9
1886	22.8	0.3	55.7	3.3	32.7	2.4
1887	22.0	2.0	56.2	2.7	32.5	2.2
1888	26.0	0.0	39.9	5.2	16.1	3.6
1889	24.0	0.2	38.2	5.2	13.8	3.5
1890	24.3	1.2	59.2	5.7	35.3	3.6
1891	31.3	1.9	57.3	4.4	32.4	2.8
1892	23.0	1.3	43.7	3.4	21.9	2.1
1893	24.0	1.5	54.9	3.6	28.8	2.0
1894	23.2	2.0	35.6	3.1	12.8	2.2
1895	25.8	1.2	48.4	2.3	20.7	1.8
1896	23.0	2.0	47.8	5.5	22.4	3.9
1897	29.5	1.7	61.2	3.1	35.3	2.4
1898	28.9	2.2	61.4	4.5	36.3	3.1
1899	22.0	2.4	57.4	3.4	32.8	2.2
1900	27.7	1.6	40.0	3.2	15.4	2.0
1901	27.5	0.8	58.7	4.2	33.2	2.8
1902	32.4	1.1	50.9	3.9	24.8	2.8
1903	28.9	0.0	53.1	4.5	28.7	2.8
1904	30.0	1.7	45.9	3.3	22.9	2.0
1905	29.0	1.0	48.2	6.5	22.0	3.1
1906	18.5	0.0	50.2	7.1	26.4	3.1
1907	35.5	0.0	65.2	7.0	41.2	3.4

TABLE 1.—(*Continued.*)

Year.	Tennessee.			
	Chattanooga.		Florence.	
	Highest.	Lowest.	Highest.	Lowest.
1867	58.6	....	31.1	....
1871	....	....	....	0.0
1872	....	....	16.4	— 0.8
1873	....	....	22.9	— 0.5
1874	....	0.7	26.0	0.0
1875	54.0	2.2	29.4	0.4
1876	31.1	1.0	19.8	— 0.6
1877	28.7	1.2	19.4	0.0
1878	19.2	0.9	13.6	— 0.8
1879	38.0	0.2	21.5	— 0.5
1880	38.3	1.0	24.5	0.1
1881	22.4	0.0	17.4	0.0
1882	40.2	1.7	29.6	1.2
1883	38.2	0.0	23.3	0.4
1884	42.8	0.2	25.2	0.3
1885	30.4	0.7	17.8	0.6
1886	52.2	1.2	28.1	0.5
1887	27.3	1.2	17.5	0.5
1888	27.0	1.8	20.8	0.7
1889	29.6	2.1	19.7	1.2
1890	42.5	2.0	23.3	1.7
1891	38.9	1.2	22.2	1.0
1892	37.9	1.1	24.0	0.8
1893	33.4	1.6	21.4	1.1
1894	25.5	0.7	17.7	— 0.1
1895	32.1	0.7	17.5	0.0
1896	40.5	1.1	20.0	0.1
1897	37.9	0.4	32.2	— 0.5
1898	24.6	1.6	13.8	0.4
1899	40.0	0.8	25.2	— 0.1
1900	24.3	1.0	19.5	0.1
1901	37.4	2.1	18.9	0.8
1902	40.8	1.2	21.7	— 0.4
1903	31.8	0.6	18.8	— 0.5
1904	22.1	0.1	17.2	— 0.5
1905	22.4	1.2	16.7	0.0
1906	33.3	3.2	16.7	1.5

TABLE 1.—(*Continued.*)

Missouri.			Connecticut.		
Year.	Kansas City.		Year.	Springfield.	
	Highest.	Lowest.		Highest.	Lowest.
1844	36.0*	....	1801	21.0	....
1873	19.3	2.0	1843	20.4	....
1874	16.2	1.5	1854	22.2	....
1875	17.8	1.8	1862	22.0	....
1876	18.0	2.0	1869	20.4	....
1877	22.2	3.8	1871	13.0	0.10
1878	19.8	3.5	1872	14.2	1.10
1879	19.2	3.2	1873	15.0	0.6
1880	16.7	?2.0	1874	17.5	1.0
1881	26.3	3.0	1875	15.0	0.8
1882	19.2	1.2	1876	17.0	0.6
1883	23.8	?5.0	1877	16.5	1.2
1884	?17.2	?3.0	1878	18.5	1.2
1885	19.1	3.8	1879	15.8	1.6
1886	15.8	0.2	1880	10.8	1.0
1887	20.2	1.8	1881	11.5	1.7
1888	20.4	4.7	1882	10.9	1.6
1889	13.9	3.2	1883	14.6	1.8
1890	17.2	0.2	1884	16.0	2.2
1891	23.1	2.5	1885	13.3	2.4
1892	24.9	1.5	1886	16.0	1.6
1893	18.1	3.1	1887	17.0	2.1
1894	20.1	4.3	1888	17.7	2.2
1895	16.9	3.3	1889	11.3	3.2
1896	19.2	2.8	1890	11.7	2.8
1897	22.8	2.0	1891	14.3	2.8
1898	21.5	4.0	1892	13.8	3.0
1899	23.3	5.2	1893	18.2	2.1
1900	17.8	4.2	1894	10.4	2.6
1901	19.4	3.7	1895	20.2	3.4
1902	23.2	3.5	1896	20.2	3.5
1903	35.0	3.5	1897	15.3	3.8
1904	25.2	2.1	1898	15.5	3.8
1905	23.0	2.0	1899	16.2	3.6
1906	19.7	2.3	1900	17.0	3.6
1907	24.0	4.1	1901	19.8	3.6
1908	30.5	3.7	1902	19.3	3.1
			1903	17.4	3.1
			1904	15.3	3.1
			1905	17.6	3.6
			1906	15.1	3.0
			1907	15.5	3.3

* Approximately.

influence. They prove conclusively that there has been no marked change since the settlement of the country began, and that such change as there has been is on the side of higher high waters and lower low waters before the forests were cut off. What the record would be if we could go back 200 years cannot be said, but it may safely be conjectured that it would show both floods and low waters that would equal or surpass any modern record. It is the experience of every engineer who has the opportunity to observe the action and study the history of great rivers to find everywhere evidence of the occurrence of higher waters than any of which he has positive record. The up-building of bottom lands, the survival of old water marks, and many other indications, show that, great as are modern floods, those of the past were greater still. In the very nature of the case, it is not possible to find similar evidence of former low waters, because such evidence is wiped out by every succeeding high water; but whoever will take the trouble to study records of early expeditions on our rivers, when barges, keel boats and similar craft were used, will conclude that extreme low water is not a modern development by any means. Measurements of the Monongahela River, at Brownsville, in 1838 and 1856, low-water years, gave discharges of 75 and 23 cu. ft. per sec., respectively. It is quite certain that the river has not fallen so low in late years. At Pittsburg in 1895 (the dryest season in recent years), it fell to 160 ft.

The point should be fully recognized that these records are valueless for establishing either side of the forestry argument unless they clearly indicate a new tendency in river flow. It is not enough to cite a few isolated cases. In a period of, say, 200 years, there must be a record year for high and one for low water. Is there any reason why it might not occur this year as well as earlier? There must be clear evidence of permanent change before any conclusion can be legitimately drawn. In two instances such a tendency may possibly be claimed, the Ohio at Pittsburg and the Connecticut at Holyoke, which show, in the past few years, a greater frequency of high waters than for some years previously.* To whatever extent this may be true, it

*In the period of 34 years from 1874, the Ohio River at Pittsburg rose above 15 ft. on the gauge 148 times. In the first half of this period, 68 of these freshets occurred and 80 in the second half. The mean for the first half was 19.3 ft. and 20.2 ft. for the second half. The mean of the lowest waters of the first half was 0.3 ft. and 1.6 ft. for the second half. In *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 31, is a 20-year volumetric record of the Connecticut, which indicates somewhat higher high waters during the last half of the period. But in this case, as at Pittsburg, higher low waters are also indicated. In fact, in both cases, the greater run-off in the later period was clearly due to greater precipitation.

is certainly not due to deforestation. The change in the forested areas on the water-sheds of either of these streams has been relatively very slight in the past 20 years. The great inroad into the timber of the Upper Ohio took place many years ago. Since that time many cleared areas have grown up to timber while new areas have been cut. The change one way or the other, in recent years, compared with the total area, is altogether insignificant. The Connecticut water-shed above Holyoke has a greater forested area than it had 40 years ago. This is due to the abandonment of former farms which, in many instances, have grown up to timber. It is doubtful if the recent cutting in the White Mountains offsets this, and, so far as snow-melting is concerned, what cutting there has been is certainly in favor of uniformity of flow.*

The records of some American rivers have been given. It is, of course, in Europe that one would expect to find more definite data because of the longer periods through which records have been kept. The histories of several of these streams have been examined without finding any confirmation whatever of the forestry theory. The floods on the River Seine, for example, show greater heights in the 16th century than in the 19th. The most exhaustive investigation of the records of European rivers, however, is that of the Danube, the great river of Central Europe, recently made by Ernst Lauda, Chief of the Hydrographic Bureau of the Austrian Government. The years 1897 and 1899 brought destructive floods to the Valley of the Danube, that of 1899 being particularly severe. M. Lauda prepared an exhaustive report upon this flood, published in 1900, accompanied by elaborate maps and tables and a searching analysis of the climatic and other conditions. In his "Concluding Remarks," M. Lauda traces the history of the Danube floods for 800 years, including in all 125 floods. His conclusions are that floods were formerly just as frequent and as high as they are in recent times, and that the progressive deforesta-

*"I have seen in the last few years abandoned farms (abandoned because of their unprofitableness) on the Western slopes of the Allegheny Mountains, which are almost impenetrable forests of thrifty trees suitable for making mine posts and telegraph poles. There are, of course, large areas subject to fires at intervals of a few years, but that they are subject to such recurrent fires is proof of their rapid production of fuel which means twigs and leaves in great abundance." (Col. Thomas P. Roberts, Pittsburg, Pa.)

"The forest area in Vermont is probably 10 per cent. greater than forty years ago. Of course the quality of the forest is inferior, but that has no effect on the water-shed." (Arthur M. Vaughan, State Forester.)

"Farms in the Connecticut Valley are among the richest in the state (New Hampshire) and have been less abandoned than elsewhere. There has been, however, a goodly acreage, very probably amounting to 25 per cent., which was cleared land in 1850, and which at the present time has reverted to forest; much of it excellent white pine forest." (Philip W. Ayres, Forester.)

tion of the country has had no effect in increasing them. In fact the records of the flood of 1899, which was a summer flood, produced almost entirely by rain, showed that it was severest on those very parts of the water-shed that were most heavily forested.

At the 10th International Congress of Navigation, held at Milan in 1905, one of the four questions appointed for discussion was the very one here under consideration. Papers were presented by representatives from France, Germany, Italy, Austria and Russia. While all the writers heartily favored forest culture, the opinion was practically unanimous that forests exert no appreciable influence upon the extremes of flow in rivers. It appears, therefore, that European experience does not support the currently accepted theory.

So much for the evidence supplied by the records in this country and abroad. The constantly reiterated statement that floods are increasing in frequency and intensity, as compared with former times, has nothing to support it. There are, it is true, periods when floods are more frequent than at others, and hasty conclusions are always drawn at such times; but, taking the records year after year for considerable periods, no change worth considering is discoverable. The explanation of these periods of high water, like the one now prevailing, must, of course, be sought in precipitation. That is where floods come from, and it is very strange that those who are looking so eagerly for a cause of these floods jump at an indirect cause and leave the direct one entirely untouched. In the records of precipitation, wherever they exist, will be found a full and complete explanation of every one of the floods that have seemed unusually frequent and severe in recent years. A few examples will be cited:

The great Kaw River flood of 1903, which wrought such havoc in Kansas City, was caused by a wholly exceptional rainfall over nearly all the water-shed of that stream. In the first three weeks of May, 1903, more than the normal amount (4.5 in.) for the entire month fell. This was followed in the next five days by 3.4 in., and upon this was piled 4.7 in. in the succeeding five days, by which time the flood had crested.

In the flood of 1906 in Western Washington, which did enormous damage and stopped railway traffic for upward of two weeks, the crest of the flood occurred about the 15th of the month. The month of October had been very wet, and the ground and forest storage was

exhausted. In the first half of November, 25% more rain fell than in the normal for the entire month, and of this about one-half came on the 13th, 14th, and 15th.

In the flood season of 1905 on the water-shed of the Upper Mississippi, there fell in the month of April above Pokegama Falls 2.55 in., in May, 4.95 in., in June, 8.03 in., and in July, 6.88 in., a total of 22.41 in. The normal for the entire year is 26.5 in.

In the record-breaking flood of 1907, in the Sacramento Valley, 88% of the normal for the month of March (based on 21 years' observation) fell in three days (17th-19th), and, on one day, the precipitation ranged from 5 to 8 in. at the different stations.

In the extraordinary flood of May and June, 1908, in Western Montana, the precipitation for May, at four selected stations, was 6.5 in., and for June, 4.2 in. The greater portion of this fell late in May and early in June. The normal for May is 2.6 in. and for June, 2.3 in.

Similar conditions prevail in every great flood, and the true explanation is found in them and not at all in the presence or absence of forests on the water-sheds. Whether the forests are in any way responsible for the precipitation itself, and so, indirectly, for the floods, brings up the third of the foregoing general propositions, viz., that forests do increase precipitation. However strong may be the popular belief in this theory, there is nothing in the records of rainfall to give it substantial support. The author has had occasion, in connection with his official work, to compare the rainfall records in the northern half of the United States from the Atlantic to the Pacific, often with this particular point in mind, and he has never found anything to indicate a change. So far as he has examined European records the same result holds, and he believes it to be true the world over except where climatic changes have resulted from causes entirely disconnected with the operations of Man in changing the face of Nature. In fact, the claim that forests increase precipitation (about 10%, according to Mr. Pinchot) leads to some contradictory results in the forestry argument. Coincident with our recent high waters, which are attributed so largely to deforestation, there has been an increase in precipitation, where there should, apparently, have been a decrease.* It is evident that, where one rule applies, the other fails.

*As a step in the crescendo of gloomy forebodings upon this subject, that have filled the periodicals during the past twelve months, the following from the September *Scrap Book* is the very latest: "When our forests are gone the streams will

So, likewise, it is held that forests are necessary to protect mountain slopes because of the greater precipitation prevailing there; yet the forests are said to increase this precipitation materially.

There is really very little, theoretically, to support the claim that forests insure precipitation. It is said that the cooler status of forest areas condenses moisture and induces precipitation; but if this were so in midsummer, when the least precipitation falls, how about the rest of the year when no such difference exists, but the reverse, if anything? Take, for example, the great forests around the source of the Yellowstone. During the period when the bulk of the precipitation falls, the temperature of the forests cannot differ materially from the outside, and it is impossible to believe that the forest exercises much influence upon the snowfall.

The fact that these high areas are generally wooded is frequently cited to prove that forests produce the higher rates of precipitation which also prevail there. But would it not be more reasonable to say that the forests flourish there because of the higher precipitation, and that the latter is due to the elevated situation and consequent lower temperature? Is not this, in fact, the reason why precipitation is nearly always greater upon the hills than upon the neighboring lowland? The mountains are Nature's wine-press by which she extracts from an unwilling atmosphere the elixir of life for the hillsides and the valleys below, and she does this whether the forests have been cut away or not.

In one respect, and a very important one, forests diminish precipitation, and that is in the deposition of dew. Dew is essentially an open-country phenomenon, where the radiation of heat from the earth's surface is unobstructed. Clouds or high cover of any kind, and also wind, interfere with this process and prevent the dew from gathering. It collects in full strength on low shrubbery, to a less degree on small trees, as in orchards, and penetrates for short distances under forest cover. In the heart of the native forest of full-grown timber, however, dew is practically unknown. The quantity deposited in the open country in a single night is quite large under

dry up, the rivers will cease to run, the rain will fall no more, and America will be a desert!" Considering how large a percentage of our forests has already disappeared, the extraordinary rains in all parts of the United States during the past year are not exactly in line with this dismal prophecy. If one were to judge from the records of the past few years only, he must conclude that deforestation is increasing rainfall.

favorable conditions, leaving the effect on shrubbery and on the ground of a considerable shower. As it gathers in greater or smaller quantities on every clear, still night in the eastern sections of the country, except in the colder season of the year, the total quantity must be quite large.*

One authority holds that dew does not come entirely from the air, but in part from the ground. It is said that water, which in the daytime passes from the ground and plants into the air, is prevented from doing this at night, because the air cannot receive it, and, therefore, it gathers in visible form on the ground and vegetation; but if this were true, it really makes no difference in the benefit which comes from the dew. Whether the low temperature due to radiation causes a deposit of moisture from the air or prevents the air from absorbing moisture which it otherwise would, the result, so far as the ground and vegetation are concerned, is practically the same.

This may be as good a place as any to note one important characteristic of precipitation, and that is its tendency to move in cycles. It is well known that dry years often follow each other for long periods with great regularity, and that these are succeeded by wet periods. Take the region of the Upper Mississippi reservoirs where the normal precipitation, based upon 21 years' observation, is 27.1 in.; in the ten years (1886-1895), this normal was exceeded only once; in the succeeding ten years the record fell appreciably below it only once. Omitting these two years, the mean for the two periods of 9 years was 24.7 and 30.0 in., respectively, an average yearly difference of nearly one-fifth of the normal. Following the well-known law that the percentage of run-off increases and diminishes with the precipitation, the disparity between the run-offs for the two periods was greater still.

This phenomenon is also admirably illustrated in the rise and fall of the levels of the Great Lakes, for these immense storage reservoirs, not only absorb and distribute annual variations of run-off, but equalize to a large degree the variations from year to year. During the period of the 80's, there was a general rise in the lake levels except Superior,

*The author has never seen any data as to the actual quantities of dew deposited in different localities and conditions, and hopes that the discussion of this paper may bring some to light. He has, however, vivid recollections on the subject when, as a lad on a dairy farm, it was his unlucky lot to go barefooted after the cows every morning without waiting to see whether the sun was going to shine or not. He knows from experience how near zero the dew point can get, and how wet dew is; and also that the warmest place in the world, at such times, is where a cow has lain all night, and next to that the dry precincts of the tall woods.

and many people ascribed this fact to deforestation, which allowed the water to find its way more quickly into the lakes. During the 90's, there was a period of general subsidence, occasioning considerable anxiety, and it was frequently asserted at that time that this was due to deforestation, which was drying up the streams. For some years now the Lakes have been rising, Ontario being the highest in 40 years; and with another wet year the levels will almost reach record heights.

The long record of the Danube floods already referred to is another example. Almost invariably high floods would follow each other for several years in close succession, and then would come long intervals of ordinary high waters.

These periodic changes are not, of course, due at all to the presence or absence of forests, for they occur just the same whether forest conditions remain unchanged or not. It is an order of Nature not at all understood, but nevertheless fully established as a fact. Just now we are in an era of high precipitation and consequently of high waters. There is a disposition to "view with alarm" these exaggerated conditions. Rarely does one stop to think how far better it is to the country to have these wet periods, even with all their floods, than the dry periods that will surely follow. A single dry year may cause more loss to the country through the shrinkage of crops than the floods of an entire cycle of wet years.

Related to the subject of precipitation is that of evaporation as affecting the quantity of water that remains upon the ground. Generally speaking the surface evaporation in summer should be greater in the open than in the forest because of the more direct action of the sun and wind; but in the height of summer the forests arrest precipitation to such an extent in the leaves and humus, that more of it escapes through evaporation than in the open. The effect of forests upon evaporation through the medium of their leaves finds its counterpart in the similar action of the growing crops that overspread deforested areas. As already pointed out, the forests of the mountains increase the evaporation from snow very materially.

Where the balance lies among all these conflicting influences affecting precipitation and evaporation it is impossible to say, and when the records are examined it must be admitted that they afford no answer. So far as the researches of Science have yet determined, the presence or absence of forests cuts no figure in climatic conditions.

FIG. 1.—DESTRUCTION FROM FRESHET IN WHITE RIVER, WASHINGTON.

FIG. 2.—NEW GROWTH OF TIMBER, ABOUT SIX YEARS AFTER FIRE.

These depend upon causes of far greater magnitude and are influenced, if at all, only to an insignificant degree by the operations of those who occupy the planet.

The fourth proposition of the forestry argument is that forests are necessary to prevent erosion on steep slopes and the consequent silting of reservoirs and watercourses below. Here again there is the same deficiency of evidence to support the theory that has characterized the three propositions already considered. The author has been unable to find anything to confirm it. In his observations, embracing pretty nearly all varieties of timber land in the northern two-thirds of the United States, he has still to see a single example where the mere cutting off of forest trees leads to an extensive erosion of the soil. Almost invariably, and it may be said always except in very unusual conditions, a soil that will sustain a heavy forest growth will immeditely put forth, when the forest is cut down (or even burned down), a new growth, generally in part different from the first, but forming an equally effective cover to the soil. The only approach to an exception to this rule that he has observed is in some of the high mountain forests where the soil is extremely thin and weak and the action of Nature in producing vegetable growth is slow. Plate CX, Fig. 2, shows one of the best examples of this class of timber land; but even here the slow growth is very evident and no large amount of soil erosion resulted. In the forest areas of the East, the growth that follows tree-cutting, consisting not only of new trees but of briars and small brush of every description, accumulates very rapidly and forms a more effective mat against erosion than the original forest itself and equally effective in storing water. Such low growths have also a better effect upon snow-melting because they give both wind and sun freer play. Certainly the ground in a forest under culture, with the débris raked up, is more easily eroded than that of a slashing or second-growth area, or even good meadow or pasture. A forest soil unprotected by forest débris is almost as erosible as a field under culture.

The increased erosion of the soil, of which so much is heard, does not result from forest cutting, but from cultivation, using that term in its broad sense to include all of Man's operations for the occupancy and utilization of the ground from which the forests have been removed. It is the "breaking of the soil" that leads to its

erosion by the elements. Roads and trails are one of the great sources of erosion in hilly countries, but plowing and tilling are the principal causes. The question is not one of forests in the first instance, but of how far the cultivation and occupancy of the soil can be dispensed with. Even on steep mountain slopes, where erosion and ruin have resulted, the effect is often due to the clumsy and injudicious work of the husbandman who uses no judgment of cause and effect in the way he exposes the soil to the force of the storms. The successful cultivation of hillsides in every quarter of the globe is an everlasting refutation of the argument that forests are necessary to protect the face of the earth wherever cultivation is practicable. Some classes of cultivated vegetation, like the well-knit turf of meadow or pasture, are a better protection against erosion than any ordinary forest cover. That there are sections of the country where erosion of the soil is much more rapid than in others under similar conditions, is perfectly true. This is especially the case with certain districts in the Southern States, and very likely forest protection is there better than any other; but it is still true that the problem of control of soil erosion on cleared lands is essentially a problem in cultivation. It is not so much the absence of the forest as it is the cutting of roads and ditches, the upturning of the soil, and the various kindred operations of Man that quicken the run-off and increase the surface soil wash.

The oft-repeated assertion that, owing to the cutting off of forests, our rivers are shoaling up more than formerly may be challenged absolutely. There is nothing in our river history to support it except in a few instances, like the Yuba River in California, where extensive hydraulic or similar operations have produced vast changes. It is exceedingly doubtful if it can be established by any evidence worthy of the name that the streams of the Mississippi basin are more obstructed by sand bars than formerly. The author's observation of upward of 20 years and inquiries from many sources fail to disclose any such evidence. It would not, indeed, be surprising if some such result were noticeable, for it would naturally seem that the cultivation of the soil has facilitated to some degree the wash into the streams. If this is the case, however, the rivers do not show it. They have a way of distributing their burdens so as to meet their

necessities and, except in rare cases, they do not shoal appreciably more than formerly.*

The distinction between erosion actually resulting from cultivation and that assumed to result from timber cutting, is important to keep in mind, for it fixes the burden of responsibility where it belongs. It shows that this erosion or soil wash can be reduced only by the elimination or control of cultivation, and the question at once becomes that of the extent to which such control or elimination is practicable. For example, it is insisted that the suggested reservoir system of the Ohio, to be referred to later on, will be absolutely dependent for its integrity and permanence upon keeping the water-sheds above them covered with forests. But it is understood not to be the policy to include in the proposed forest reserves any lands that are fitted for agriculture.† As elsewhere pointed out, that portion of these areas, which is not reduced to cultivation, will not be subject to erosion more than at present by the mere fact of cutting off the timber; for the natural growth on logged-off lands is just as good a protection as the forests themselves. If the agricultural tracts are still to be left open for occupancy, the source of sediment remains uncurbed and the whole argument for forest reserves, on the ground of protecting the reservoirs from sedimentation, falls to the ground.

Some reference should be made to the real significance of the alarming reports which have been put forth concerning the washing of our soils into the sea. Over and over during the past year has the statement appeared that 1 000 million tons of our soil is annually carried by our rivers into the ocean. This figure itself is quite conservative, but the conclusions drawn from it are not at all so. Taking the results of silt observations on the Mississippi River and its tributaries for 1879, and applying the Missouri rate to all Western streams outside the Mississippi basin, and the Ohio rate to all Eastern streams outside the same basin, a total of about 1 100 million tons is indicated.

*The absurd length to which this erosion argument has been carried is well illustrated by the remark made in a recent address by one of the officials of the Forestry Service: "This energy (of running water) is expended in rolling along stones and gravel to finally build up the mouths or beds of the great rivers. Next year there will be a bill introduced in Congress providing a forest reserve in the Appalachian Mountains, so that the rocks from these mountains will be kept from the Mississippi River!"

†Among references to the intention not to absorb agricultural lands in the areas conserved by the reservoirs is the following from A. F. Horton, Assoc. M. Am. Soc. C. E., in *Engineering News*, June 11th, 1908: "The reader should not lose sight of the fact that the conserved area is not rendered unfit for cultivation or other use, but that only a small portion of the conserved area (that covered by the reservoir) is so utilized that its value for cultivation is destroyed."

But 1879 was a low-water year in the Mississippi basin and the quantity for average years may probably be 1 500 million tons and for extreme years, 2 000 million tons.

Let us look these prodigious quantities squarely in the face and see what they mean. Where does this enormous volume of soil come from? Is it, as one might infer from published references to the subject, from our cultivated fields—an annual toll laid upon the precious fertility of our agricultural lands? Not at all. Only a very small proportion comes from this source. Possibly half of the total quantity of sediment goes down by the Mississippi. All authorities agree that the greater portion of this comes from the Missouri. From computations which the author has made, he believes that fully two-thirds of it comes from that source. The observations of 1879 indicate that five times as much sediment comes from that stream as from the Ohio. But where does the Missouri get it? Almost entirely from the most useless areas of land with which any country was ever afflicted. The barren Bad Lands are the principal source. Much comes from the mountains; much from the sand hills; very little, relatively, from cultivated areas. Of the balance of the soil wash of the United States, by far the greater portion comes from other similar sections of the West where the streams carry enormous loads of sediment. The entire Colorado system is even more distinguished in this respect than is the Missouri. The same is true of the Rio Grande, the Pecos and the upper courses of the Arkansas and Red. Even the streams of the Great Interior Basin are heavy silt bearers, and the same is true of many of the streams of the Pacific Coast. The streams flowing into Puget Sound are heavily laden with silt at certain portions of the year, and the great Columbia Bar is impressive evidence of the vast burden of sediment which that mighty river has carried to the sea. Nearly all of the annual load carried by these streams is entirely unaffected by anything which Man has done. It is the regular natural carving down of the hills and building up of the valleys and estuaries below.

The Eastern streams are clear and sediment-free compared with those of the West; but even in these, a large portion of their sediment is eroded from the gorges and cañons of the hills and mountains, which will continue to wash away as long as the rivers flow. This particular class of erosion, on both Eastern and Western rivers, is far less objec-

tionable than one is led in these later days to believe. Has it not from the beginning been one of the most beneficent operations of Nature? Are not the richest lands in the world—the river bottoms and deltas—built up in this way? To a very great extent the irrigated lands of the West are composed entirely of the débris from the mountains and the bad lands. Even to-day this tribute from the highlands is of great value. The periodic enrichment of the Ohio bottom lands and similar tracts in hundreds of other places, is of the highest economic importance. The soil-laden waters of irrigation in the spring, though sometimes injurious to the growing crop for the time being, are, on the whole, extremely beneficial. The damage from sediment is not in its injury to the lands ordinarily, but to ditches, canals, reservoirs and similar works. On the whole it is, and always has been, a benefit to the lowlands. Even that portion carried out to sea builds up deltas and surely, though slowly, extends the habitable area of the globe. Not alone in the resources of water and timber, but in the perpetual renewal of soil as well, has the valley said to the mountains throughout the world's history: "I will lift up mine eyes unto the hills from whence cometh my help."

Sediment of this character, except when accompanied by alkaline salts or other similar ingredients, is not injurious to domestic supply. The water of the Missouri River is one of the healthiest drinking waters in the world in spite of the fact that it is one of the muddiest.*

The proportion of soil wash that comes from cultivated fields is really very small compared with the enormous total that the rivers carry away. Heavy rains undoubtedly wash farm soils a great deal, but this erosion is in large part a transfer from one spot to another and not an absolute loss. The history of the old Ohio Canal reservoirs indicates very little filling in the 66 years that they have been in existence. According to the Chief Engineer of the Ohio State Board of Public Works, it is scarcely appreciable in some of the reservoirs and in none does it amount to as much as 6 in., or $\frac{1}{200}$ in. per year from the tributary water-shed. Yet these reservoirs are surrounded by rich agricultural lands. The silt observations on the Ohio in 1879 indicate only a little more than $\frac{1}{600}$ in. over the entire water-shed; but this, it is true, was a year of light rains.

*The late J. B. Johnson, M. Am. Soc. C. E., used to say, in extolling the virtues of Missouri River water, that it was the most perfectly filtered water in the world; with this difference, however, that in the ordinary case water is run through the filter, but here the filter is run through the water.

It is readily seen that the formidable danger of which so much has been written of late becomes quite harmless as to quantity when it comes down to the individual farm. The harm is probably not so much in the quantity of soil actually lost as in the fact that the soil may be leeched of some of its more important ingredients. The evil is one which can be controlled only by better methods of farming, whereby the surface waters will be restrained from eroding the soil; but even these measures have their adverse side, for when heavy rains prevail for a long time it is more important to the farmer to get the water off his land than it is to save a little soil. Most of the soil will stop on lower ground and not be wholly lost, but if the water is not gotten rid of the crop may be ruined.

The caving of the banks of our great rivers is constantly cited as an example of soil loss on an enormous scale, and it is asserted that this condition is worse now than formerly. The Mississippi and Missouri Rivers, practically alike in this respect, are the two most prominent examples. The author will consider briefly the case of the Missouri because he has had a long and intimate acquaintance with that stream from its mouth to its source.

It may be stated by way of refutation that the actual condition of this stream to-day is better than before settlement began in its valley, except that possibly the low-water flow is slightly diminished to meet the demands for irrigation. The stream is not "constantly becoming more and more savage," as a recent writer asserts. On the other hand, its natural savagery is much restrained. Probably 100 miles of its banks are protected; snags and drift heaps are largely removed; considerable bottom land has been reclaimed and turned to industrial use; floods are no greater than they used to be, and navigation is safer and easier. Navigation has ceased, not because the river has deteriorated, as is commonly asserted, but because the natural difficulties peculiar to this stream are so great and so hard to overcome that boats cannot live and do business at the same rates at which railroads transport freight.

That the river is a most destructive one to the bottom lands along its course is only too true; but the character of its destructive work is generally misunderstood. The writer just quoted states that the river carries away annually 8 000 acres of bottom land within the limits of the State of Missouri alone. The total acreage of these lands

is about 640 000. If this statement were true, more than the entire area would have been carried away since the voyage of Lewis and Clark, and if the process had been continuous since Columbus discovered America, the river to-day would be flowing in its original channel in the solid rock, 75 to 90 ft. below the present surface. As a matter of fact, there is more soil in the valley to-day than there was at the date of either of these events. Taking an average for a considerable period, none of the bottom land is lost. It has always been slowly rising through accretion. The bank caving is only a transfer from one point of the shore to another. For every dissolving bank there is a nascent bar. Where steamboats ran last year, willows may be growing this, and next year the farmer may be planting his corn. The havoc wrought concerns the individual owner, but not the valley bottom itself. The cruel losses attract attention; the unobtrusive gains do not; but the account always balances itself. The harm done is first to the individual whose possessions are swept away, and second to the community through paralysis of development, depreciation of values, and the holding back of this natural garden spot from becoming what it ought to be. The evil is a very real one, and the author has long endeavored, though without success, to secure provision in the River and Harbor Bill for its amelioration.* Great as the evil is, however, it is not at all in the nature of an actual loss of land to the valley.

It must be clear from the foregoing that the bottom lands of the Missouri add nothing whatever to the total quantity of sediment that passes out of the mouth of the stream, for these bottoms have been increasing rather than diminishing in quantity. Likewise the Mississippi bottoms contribute nothing to the volume of sediment that is carried into the Gulf of Mexico. It all comes from the uplands, far and near, but principally from the more remote and hilly regions. This load is in the nature of through traffic. The local freight picked up from a caving bank is mostly discharged at the next station. It follows, therefore, that if the banks of these streams were revetted from the Gulf to Pittsburg, the Falls of St. Anthony and the mouth of the Yellowstone, the quantity of sediment passing into the Gulf would not be diminished a particle. Such revetment would nevertheless be of the very highest value, if it could be made to hold.

Transactions, Am. Soc. C. E., Vol. LIV, p. 336.

for it would give permanence to the banks, security to riparian property, and would largely prevent bar-building by training the river in a regular channel and relieving it of everything except its through load of sediment.

The bank-caving problem of these valleys is unaffected in any appreciable degree by the influence of forests or cultivation on the water-sheds, and cannot be solved or materially assisted by any practicable changes in these conditions. The problem is strictly a local one, and the remedy must be a local one. Even if it were possible to bring the waters down from the uplands perfectly clear, it is not at all certain that the effect upon the bottom lands would not be injurious rather than beneficial; for then the caving soil, instead of being quickly deposited again, would in part be carried out to sea, and the bottom lands, unless protected, would be gradually eaten away.

In addition to the four main propositions discussed above a few subordinate features of the question will now be considered.

A feature of the Forestry Service which is generally overlooked is the possible effect of culture upon the bed of humus so much relied upon in these discussions to prove the restraining action of forests upon run-off. Mr. Pinchot, in his statement to the Judiciary Committee, said:

"The effect of a forest on a steep slope is to cover that slope with leaves, rotten and half rotten sticks, and other mechanical obstructions which prevent the water from running below as rapidly as it would otherwise."

It is understood that the forest policy is to keep this litter cleared up as a measure of fire protection, and one frequently sees in articles on forestry, photographs of the typical forest culture in which the ground is thoroughly cleaned up. The result must be to diminish proportionately the retentive action of the forest bed and to increase its liability to erosion. In the light of the foregoing discussion fire protection is of much greater importance than the retentive effect of the forest bed on the run-off. The remarkable degree to which the forest bed will dry out in prolonged drought, making it one vast tinder-box, supports this conclusion, and is another proof of the extreme desiccating effect of forest growth upon the soil.

FIG. 1.—CUT-OFF ON MISSOURI RIVER ABOUT TO TAKE PLACE.

FIG. 2.—CREW FELLING TREE FROM RIVER BANK.

It often escapes attention, except with those who are in the woods a great deal, that the water establishes little channels through the débris where the latter is of long accumulation and somewhat permanent in character. Such débris does not in reality offer so great an obstruction to flow as one would suppose, and as would be the case if its condition underwent frequent change.

The statement is constantly met that forests are very efficacious in the protection of river banks from undermining and steep slopes from sliding. The exact reverse is the case. As every river engineer knows nothing is more disastrous to a river bank on an alluvial stream than heavy trees. This is due partly to the great weight, but in large part to the swaying effect of the wind and the enormous leverage of the long trunks which pry up the ground and facilitate the tendency to undermining. One of the regular policies of river control is to cut down these trees for a distance back from the edge of the bank wherever complications with private ownership do not prevent. Snags and drift-wood in the channels have always been among the most serious obstacles to navigation on streams flowing between forest-covered banks. Likewise where railroad or highway grading cuts the skin of unstable mountain slopes, the presence of large trees immediately above tends powerfully to loosen the ground and cause it to slide; and in such cases it is necessary to cut down the timber. Far better than forest trees on river banks are thick growths of willow, alder, or any of the smaller close-growing shrubs; and on side hill slopes either such shrubbery or a good turf.*

In the current discussion a great deal is made of the fact that mountain slopes are "quick spilling," the deduction being that they therefore are more productive of floods. This is quite contrary to the fact. It is perfectly true that more rain falls on the hills than on the lowlands, that a greater percentage of rainfall runs off from steep than from flat slopes, and that it runs off more rapidly; but it does not follow at all that these conditions produce greater floods. A mountain stream carries off the water within its banks a great deal faster

*The following testimony before the Board of Consulting Engineers, Panama Canal, is to the point (Report, p. 329):

Question by Mr. Welcker: "Mr. Chairman, I would like to ask if Mr. Dauchy thinks that vegetation prevents the sliding?"

Mr. Dauchy: "My experience has been the reverse; I have stopped sliding hills by cutting off the vegetation. The weight of the timber on a sliding slope aids materially to assist the sliding."

Mr. Welcker: "Does not the vegetation diminish it?"

Mr. Dauchy: "If you could get a grass-covered slope it would help to diminish it."

and more safely than a similar stream in the lowlands. The banks are almost always stable and the bottoms rocky or composed of heavy gravel or boulders; in fact, floods do less harm on such streams than on any others. In the low land, where the streams have smaller slopes and unstable banks, much smaller run-off produces greater floods and more destruction. Moreover, Nature to a large degree adapts streams to the work required of them. The channels of the tributaries of the Ohio have been carved out through long ages to carry in safety the average flood flow. Area for area of water-shed, their cross-sections are much larger than those of streams in climates of less rainfall. The normal section of the Ohio at Wheeling is over 2 sq. ft. for every square mile of water-shed, while that of the Kaw River at Kansas City is less than $\frac{1}{3}$ sq. ft. per square mile. It is therefore wholly erroneous to conclude that the streams of these mountains are more subject to over-bank freshets than those of the low lands or that the freshets themselves are more destructive. Considering the conditions growing out of settlement the reverse is unquestionably the case.

There is one other consideration of prime importance in this forestry argument, and that is the fact that no possible development of forestry can increase the present percentage of forest-covered areas. At least as much ground as is now devoted to agricultural purposes must continue to be so used. The utmost admissible expansion of National forests will never require a greater area than is now occupied by forests and second growth or logged-off lands, which, so far as run-off and erosion are concerned, are just as effective as the virgin forest itself, and more effective than will be the groomed forest of the new régime. There may be a shifting of areas devoted to forests, but possible expansion, compared with the present area, is so small that its influence upon the great rivers, even admitting the full force of the forestry argument, would be wholly inappreciable.

The fact just dwelt upon should make us thankful that the forestry theory as to the stream flow is not correct. Whatever the value of forests we cannot have them everywhere, and by far the greater portions already cleared away must always remain deforested. If this fact of deforestation has brought with it in greater degree than of old the calamities of high and low waters, then, indeed, we are in an unfortunate case. But it has not done so. Nature has decreed no such penalty for the subjugation of the wilderness, and on the whole these

natural visitations are less frequent and less extensive than they were before the white man cut away the forests.

In summarizing below the foregoing argument, the author would be particularly careful to guard against sweeping assertions in any of his conclusions. He well understands how little the subject is capable of precise demonstration. Snow, for example, does not always fall, even in the open country, under the influence of the wind, or it may fall in a wet condition that keeps it from drifting. Altitude comes in with its lower temperature and modifies the general result. There is a vast difference between a northern and a southern exposure even with the same slope and topographical conditions. Precipitation scarcely ever occurs twice alike on the same water-shed. The combination of flow from tributaries is never the same in any two floods, and there is an endless variety of conditions that must qualify our rules and make us cautious in making claims in a matter of this kind. The author objects solely to the contrary course pursued by many forestry advocates—to the extreme claims that forests exert a regulating influence upon stream flow in times of great floods or extreme low water in our larger rivers. These claims stand to-day absolutely unproven. The difference between past and present conditions is not great. One influence offsets another with such nicety that the change, if there is any, is hard to find. The "delicate balance" maintained by Nature where Man has not cut away the forests is replaced by other balances equally delicate and efficacious in the drainage of lands, the growing of crops and the deposition of dew.

In the following seven propositions the author sums up the arguments presented in the foregoing pages:

(1) The bed of humus and débris that develops under forest cover retains precipitation during the summer season, or moderately dry periods at any time of the year, more effectively than do the soil and crops of deforested areas similarly situated. It acts as a reservoir moderating the run-off from showers and mitigating the severity of freshets, and promotes uniformity of flow at such periods.

(2) The above action fails altogether in periods of prolonged and heavy precipitation, which alone produce great general floods. At such times the forest bed becomes thoroughly saturated, and water falling upon it flows off as readily as from the bare soil. Moreover, the forest storage, not being under control, flows out in swollen

streams, and may, and often does, bring the accumulated waters of a series of storms in one part of the water-shed upon those of another which may occur several days later; so that, not only does the forest at such times exert no restraining effect upon floods, but, by virtue of its uncontrolled reservoir action, may actually intensify them.

(3) In periods of extreme summer heat forests operate to diminish the run-off, because they absorb almost completely and give off in evaporation ordinary showers which, in the open country, produce a considerable temporary increase in the streams; and therefore, while small springs and rivulets may dry up more than formerly, this is not true of the larger rivers.

(4) The effect of forests upon the run-off resulting from snow-melting is to concentrate it into brief periods and thereby increase the severity of freshets. This results (*a*) from the prevention of the formation of drifts, and (*b*) from the prevention of snow-melting by sun action in the spring, and the retention of the snow blanket until the arrival of hot weather.

(5) Soil erosion does not result from forest cutting in itself, but from cultivation, using that term in a broad sense. The question of preventing such erosion or soil wash is altogether one of dispensing with cultivation or properly controlling it. The natural growth which always follows the destruction of a forest is fully as effective in preventing erosion, and even in retaining run-off, as the natural forest.

(6) As a general proposition climate, and particularly precipitation, have not been appreciably modified by the progress of settlement and the consequent clearing of land, and there is no sufficient reason, theoretically, why such a result should ensue.

(7) The percentage of annual run-off to rainfall has been slightly increased by deforestation and cultivation.

If the foregoing propositions are correct they enforce two very important conclusions—one relating to the regulation of our rivers and the other to forestry.

It follows that no aid is to be expected in the control or utilization of our rivers, either for flood prevention, navigation or water power, by any practicable application of forestry. Remember always that it is the extreme of flow, not the medium condition, that controls the cost of river regulation. It is the floods and low waters that measure the cost. Any scheme of control that is not based upon these

is worthless. This proposition need scarcely be urged upon the experienced engineer. For himself he would never place any real reliance upon forestry. Called in consultation, for example, in the problem of protecting the City of Pittsburg from floods he would be bound to take as his measure of the problem the highest recorded flood on the river with a good factor of safety on that, and then figure out by what methods—artificial reservoirs, levees, raising of grades, or clearing the river channel of artificial obstructions—he would obtain the desired relief. He would not dare, as the physician in the case, to advise his patient that he could dispense with or lessen in any degree the application of the remedies proposed, nor save one dollar of the cost, by anything that might be done in reforesting the water-shed of the rivers themselves.*

In like manner no engineer could honestly advise lowering in height by a single inch the levees of the Mississippi because of any possible application of forestry to the water-shed of that stream. And again he could not advise that forestry development would lessen in any degree the cost of improving the rivers for low-water navigation. Engineers fully understand their responsibility in these matters. But great engineering projects cannot be carried out without money, and the people will not give the money unless convinced of the necessity and wisdom of the plan proposed. So long as there is apparently some easier and simpler plan, some panacea, no matter how nebulous or unproven, that offers a way out without the expenditure of so much cold cash, they will be backward in voting money, and the counsel of the engineer will be of no avail. Hence the complete divorcement of forestry from any connection with river regulation—so far, at least, as its effect upon the cost of such regulation is concerned—will be a distinct and positive gain to the latter.

In the second place, forestry will be left to work out its own salvation without any reference to the rivers. Will not its cause be promoted by this divorcement? At first thought it may seem that thereby one great argument for forestry is lost; but no argument can be of

*Possibly the author is too positive in this opinion. He finds that, in one case at least, the City of Williamsport, Pa., reputable engineers have advised reforestation of mountain slopes as a protection against floods. The statement of "an eminent authority" was cited with approval to the effect that "four-fifths of the precipitation is detained by the surface of the ground" under forest cover. But here, as in all these assumptions, the rule applies only to the average condition. The point is overlooked that in periods of heavy precipitation the retentive capacity of the forest bed becomes exhausted. If the City of Williamsport is relying upon this advice it is certainly laying up for itself a season of repentance.

value in the long run that is not based upon truth, and the disappointment that is certain to result in the fulfillment of these hopes will do more harm than good. Forestry does not need any such support. It stands on a basis of its own, too broad and too sure to require any extraneous aid. What is this basis? The reply may be given in the beautifully appropriate phrase that occurs in the Act of Congress creating the first of our National Parks, "the benefit and enjoyment of the people." In the matter of benefits, forests are necessary, because they produce the most important material of construction known to Man; even iron cannot be excepted. From the lead pencil to the mast of a ship, from the infant's top to spacious temples and palaces, it enters into nearly every requirement of human existence. A large portion of the structures for human habitation are built of it. The land transportation of the world is closely dependent upon it, for if it were not for the railroad tie scarcely a car could run. It is only when one stops to think a little upon the unlimited adaptability of wood to human needs that its transcendent importance is borne in upon him.

In the matter of enjoyment, no other work of Nature has done more for the uplifting and ennobling of the mind than these "first temples" of God. It requires no argument to enforce this assertion, particularly with him who has been reared in close companionship with the woods. Sad indeed will be the day, if it ever comes, when the people are deprived of this source of healthful pleasure for which no adequate substitute can ever be found.

And yet this supremely important resource in human happiness is strictly limited, and the visible supply is fast disappearing. Statistics fix the date, almost as confidently as an astronomer predicts an eclipse, when the doomsday of its final disappearance will come unless something is done to prevent. Most fortunately this material, unlike copper or iron or stone, is a vegetable product capable of self-renewal, and the supply can be kept up forever. This is what gives its extreme importance to forestry. It requires no dubious support from any other source. It fully justifies the splendid work that the Forestry Service is doing and demonstrates the wisdom of the far-sighted men who are laying the foundation of our future National forests.

Let us now inquire if it will not be to the advantage of this great work to be absolutely independent of any connection with water-

way development. Will it not be better in every way for forestry if it is promoted solely on the basis of producing trees for human use and enjoyment, and not at all for any supposed influence upon flow of streams? Is it really a wise move, so far as forestry is concerned, to single out the rugged and inaccessible mountains as localities where our future supply of timber must come from? The availability of forests to human needs depends very largely upon the situation in which they grow. Few people understand the exceeding importance of this matter. The converting of a forest tree into form for use involves two distinct processes, the conversion of the tree into lumber or other product and its transportation to the place of consumption. The cost of logging operations is immensely increased by the roughness of the ground. In our Western forests, for example, it requires a higher grade of skill, commanding higher wages, to "lay" a tree on a steep hillside than on even ground. The losses from breakage in falling are much higher, and the difficulty and expense of getting the logs out much greater. In fact, the increase of cost runs all the way from $1 to $10 per 1 000, depending upon the situation. *Engineering News* stated the case very forcibly in regard to the Appalachian forests (though it did not have this particular thought in mind), when it said in a recent issue that "the cutting off of forests on the remote mountain slopes has only become possible with the high price of lumber that has prevailed for ten years past." This increase of cost represents the perpetual tax that the public must pay for timber from these regions as compared with that from the lowlands. And a great deal of it can never be gotten out at all. The poet's "gem of purest ray serene" was not more lost to human needs than are tens of thousands of noble trees in the rugged fastnesses of our mountains, east and west. Benefit? To convert them into lumber will cost more than they are worth. Enjoyment? Only the solitary hunter or mountaineer ever sees them. These are not the places to rear up forests for the good of the people.

Consider the question of transportation and take Chicago as being practically on the meridian through the center of population of the country. The rate on fir from the Cascades to Chicago is 55 cents per 100, or $16.50 per 1 000 ft. B.M. The average rate from the Appalachian forests is about 18 cents, or about $9 per 1 000 for green oak. By a proper distribution of our forests these rates on the average

ought to be brought within 10 cents per 100. In logging and transportation together, the country will tax itself on the average not less than $10 per 1 000 for whatever supply it derives from these mountain forests as compared with what it might receive from forests more favorably located.

If it were not for the erroneous assumption that forests have a regulative effect upon the flow of our navigable rivers, would not the policy in regard to the acquisition of lands for forest reserves be quite different from that now proposed? If Congress were to vote, say, $10 000 000 at the next session to commence the establishment of National forests by purchase, would it not be far better spent in lands where the pine, oak, cherry and ash used to grow, in locations convenient for access by the people and in every way better adapted to their needs? States, counties or other agencies should be required to meet half the original cost. Even if the total cost to the Government were several times what equal areas in the mountains cost, it would be far more economical in the long run. There is an abundance of land in nearly all the States, suitable for the purpose, that can be had at not excessive cost. In New England, for example, would not the development of forests in the lowlands, where in many places former cultivation has been abandoned, be far better than to buy up the difficult slopes of the White Mountains? Let there be a National forest in every county of the United States where it is practicable to create one. Let its location be carefully chosen so that its product may be manufactured and shipped with the smallest cost to the people, and serving also, not only as a pleasure ground, but as a stimulus to similar work by private agencies.

It will be urged that these mountain lands are worth more for forestry than for agriculture. Very true, but that would not justify their purchase if the same money would produce a better result elsewhere. "Never buy what you do not want because it is cheap." Again, it may be said that here is our only remaining timber supply in the East, and it must be saved. Except in some possible economy by the more judicious cutting under Government control, it is not apparent how a forest tree that has attained its growth is going to render any greater good to humanity by being saved for the next generation than by being cut for this. There is a general sentiment current in these later years that if timber is cut off by private agencies it is wasted;

but does it not find its way into common use just the same? Not as completely, perhaps, but still substantially the same. Take the combination of the Weyerhauser Timber Company, considered entirely apart from its economic and ethical aspects as a great trust or corporation, and solely as a preserver of our forests. With its system of fire control, its policy of holding its timber for high prices, is it not really conserving the timber for future use? To speak of such timber as being "lost" to the people, "wasted," and its acquisition as a "looting of our heritage," is as disingenuous as it is untrue. Will its lumber cost the consumer a cent more per thousand than if it were from a Government reserve? It is a wholly gratuitous assumption that our timber is going to be "wasted" unless it is placed under Government control. The thing of prime importance is to get new forests started. In the 30 to 50 years that our present supply will last, new forests should be brought into existence all over the country. This is far more important than to buy the virgin timber of the Appalachians.

Moreover, it seems now to be considered that the virgin lands have already risen too high in price to be purchased by the Government, and that it is only the second-growth lands that can be economically acquired.* Be that as it may, it is certain that the acquisition of such of these lands as are desirable for the strict purposes of timber production will be greatly facilitated by disabusing the minds of the owners of the impression so diligently fostered of late that the very salvation of the country depends upon their selling out to the Government. Can anyone doubt that the present course will add vastly to the purchase price?

Still another argument that may be urged is that only by linking the forests with the rivers in a way to establish their utility in maintaining navigation can the constitutional objection to the acquisition of these lands be overcome. But does this apply to mountain forests more than to any others? It is incontestably true that whatever restraining effect forests have upon run-off is greater upon the lowlands than upon steep mountain sides. This legal feature of the question will be referred to farther on.

*Report of Secretary of Agriculture on Southern Appalachian and White Mountain water-shed, December, 1907, pp. 8, 30, 35.

Reservoirs in Their Relation to Stream Flow.

Under this heading artificial reservoirs alone are included. Natural reservoirs of various kinds exist nearly everywhere and exert a profound influence upon stream flow. The ground is the most important of these, absorbing on the average probably one-third of the total rainfall. Natural lakes are great regulators, the St. Lawrence system being the most perfect example. Forests are effective reservoirs at certain seasons. Swamps and low-lying grounds along river courses, like the great flood basins of the Sacramento and the Mississippi, are, in their natural state, enormous reservoirs which greatly reduce the flood flow of the river channels. Snow-drifts, particularly the great drifts of the mountains, are splendid reservoirs. The streams themselves have immense storage capacity; for example, the Mississippi within levees stores at least 2 000 billion cu. ft. of water from Cairo to the Gulf, between extreme high- and low-water stages. All these reservoirs and many of less importance are ever active in regulating the flow of streams. Without them precipitation would flow off as fast as it arrives and our greatest floods would be magnified many times.

Here we are considering only those reservoirs constructed by Man to supplement and extend the regulating effect of Nature's reservoirs. If the conclusions reached in the first section of this paper are correct, forests cannot be relied upon in any degree to help solve the problems of high and low water. Present conditions must be met by purely artificial means, since Man has so far discovered no way of controlling the climatic conditions which govern precipitation. He cannot "stay the bottles of heaven" in times of flood, nor open them in seasons of drought. He must take the water after it reaches the earth and deal with it the best he can.

The artificial reservoir is intended to attack this problem at its source. It catches and holds back the water in the near vicinity of its deposition, instead of waiting until it gathers into the rivers and then building huge bulwarks to contain it there in times of flood. It saves the stored-up supply and gives it out in the low-water season, thereby helping navigation, instead of dredging and otherwise treating the watercourses to increase the low-water depth. It corrects one of the greatest deficiencies of Nature by abolishing inequalities of stream flow and converting waste into utility. Theoretically, it is

the perfect plan. It has always appealed to the imagination of layman and professional alike. It has often been resorted to, and the number of reservoirs in the world is very great and constantly increasing. Hitherto they have been mainly used for power, municipal supply, irrigation, and for navigation in canals. In very few instances have they been applied to improve the navigation of large natural watercourses, and in none, so far as the author is aware, for the exclusive purpose of preventing floods.

The question arises, why are they not regularly applied to these last-mentioned purposes? The answer may at once be given that in the general case the cost is greater than the benefits to be received. This element of cost arises mainly from the absence of good sites (including dam sites as well as holding basins) and also, to considerable extent, from an interference with the purely artificial conditions growing out of the settlement of the country.

The best reservoir site is a natural lake. Such a site is already covered with water, and original conditions are not materially changed. Evaporation is not much increased by the necessary enlargement. Smaller and safer dams accomplish a given storage than for the average dry site. The question of public health involved in uncovering large areas for reservoir beds in the heated portion of the year is less serious. Everything makes these sites the most advantageous that can be found, and it may be laid down as a rule that the public good requires the utilization of every such site to the fullest possible extent.*

Except in a few cases, dry sites are deficient in these advantages. Greater areas of land have to be condemned, and larger and costlier dams are required, with vastly greater danger in case of accident. Really good sites are not as abundant as one might wish, and the

*An interesting feature of these natural reservoirs may be noted. A natural lake, wholly uncontrolled at its outlet, may have a more effective control of the outflow than an artificial reservoir of equal superficial area when full, though of far greater capacity between high and low water. The outflow from a lake can be increased only by storing simultaneously a quantity of water measured by a rise in the surface equal to that in the outlet necessary to give the increased flow. But if the artificial reservoir has reached the limit of its allowable filling, the outflow must be made equal to the inflow. If this limit is reached before or at the time of maximum run-off, then a quantity equal to this run-off must be let out of the reservoir. This contingency can never happen in a natural lake. The turning point where outflow and inflow balance each other is always after the crest of the flood has passed—in fact at the time when the diminishing inflow and increasing outflow balance each other and the lake ceases to rise. In the case of the Yellowstone Lake (140 sq. miles), for example, this rise, in average seasons of snow-melting, continues from 10 days to 3 weeks after the inflow has reached its maximum, and surrounding streams have subsided materially before the Yellowstone River (at the lake outlet) ceases to rise.

problem of developing storage on such sites is beset with difficulties of many kinds that greatly increase the cost.

In 1897 the author made a careful study of this question of flood control by means of reservoirs in connection with an official investigation of the advisability of building reservoirs in the arid regions. His view of the difficulties in the way of any general application of such a system is quite fully stated in his report,* and the following extracts are directly in point:

"It is the cost, not the physical difficulties, which stands in the way. It may be stated that as a general rule a sufficient amount of storage can be artificially created in the valley of any stream to rob its floods of their destructive character; but it is equally true that the benefits to be gained will not ordinarily justify the cost. The reason for this is plain. Floods are only occasional calamities at worst. Probably on the majority of streams destructive floods do not occur, on the average, oftener than once in five years. Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone. The cost, coupled with the loss of so much land to industrial uses, would be far greater than that of levees or other methods of flood protection. * * * The construction of reservoirs for flood protection is not, therefore, to be expected, except where the reservoirs are to serve some other purpose as well."

The above conclusions are still as applicable as they were when written. The subject has been given renewed prominence quite recently in connection with the Ohio River floods, but, before considering this particular application, attention will be given to certain reservoir systems that have been proposed elsewhere and particularly to one already built and put in operation by the Government and which will be referred to frequently in the following pages. This is the system at the head-waters of the Mississippi—the largest artificial reservoir system in the world.

The project of converting the more important of the numerous lakes around the sources of the Mississippi and its tributaries into

*House Doc., No. 141, 55th Cong., 2d Sess., p. 46.

FIG. 1.—LAKE WINNIBIGOSHISH DAM.

FIG. 2.—PINE RIVER DAM.

storage reservoirs as an aid to navigation was originally proposed by General G. K. Warren, and was first put into definite shape by Colonel Farquhar, of the Corps of Engineers. The plan then embraced a large number of lakes in Minnesota and Wisconsin, but only five sites have actually been improved. The dams were first built of timber cribs, but have recently been rebuilt in concrete, as shown on Plate CXII. The combined storage is about 93 billion cu. ft. It is about twice the mean annual run-off from the water-shed, and the system is probably the only one, except the Great Lakes, which equalizes periodic as well as annual fluctuations of flow. That is, it carries over the surplus from wet years to help out in dry years, and its utility is, therefore, of the most comprehensive character. The cost of the five reservoirs is remarkably low, although it is not now possible to tell the exact cost of the present structures on account of the mixture of old and new work; but it probably does not exceed $750 000, including a lock in the Sandy Lake Dam. This is only $8 per 1 000 000 cu. ft., or 35 cents per acre-foot on the basis of total capacity. It would be about twice this on the basis of the mean annual run-off from the water-sheds.

A large portion of the original project has been abandoned because public sentiment did not support its continuance. The author has always regretted this backward step, as he believes in developing to the fullest extent the exceptional opportunities here offered for the storage of water. The available reservoir sites which could be cheaply improved in Minnesota and Wisconsin are sufficient to control absolutely the floods of the Mississippi within the danger line, for a long distance below St. Paul, and to improve the navigation of the upper river very materially, while their value for industrial purposes is almost beyond estimate.

In spite of the great and obvious advantages of this system, it has not yet received the popular approval that might be expected of it. In fact, about three years ago, there arose a widespread sentiment in the community around the reservoirs that the system was, on the whole, injurious, that its disadvantages far offset its advantages, and a strong movement was organized to have it abolished altogether. For the purpose of investigating this matter, a Board of Engineers was appointed, of which the author was a member. The Board found that there was a general belief among the people below the dams that

they actually increased the floods, while the people above complained bitterly of the back waters caused, throughout that low country, by filling the reservoirs so full. The water powers immediately below the dams complained that they were not getting even the normal flow of the stream, which was the case. Navigation interests below St. Paul have always been lukewarm in regard to the beneficial effects of the reservoirs, and the Board was able to find only one steamboat captain who would make a positive statement that the boating interests derived any particular benefit from them.

Some curious results developed in this investigation. It was found that, great as the reservoirs are, conditions may arise in times of excessive precipitation that will compel them to discharge a greater quantity of water than would flow from the lakes in their natural condition. That is, they might actually operate to increase the floods if they should fill to their limit during a period of excessive precipitation. This very contingency nearly happened in the season of 1905, as may be seen on the curve (Fig. 2) illustrating the effect of the reservoirs on the flood situation at Aitkin, Minn., about 130 miles below. On Fig. 3 is shown a comparison of the natural and regulated flow of the Mississippi below the three upper reservoirs, the latter being operated in the interest of navigation alone. These curves show very clearly that the natural flow of the river was more uniform than is the regulated flow.

In like manner, during the period of lowest water, viz., in midwinter, the reservoir gates are closed down to about 400 cu. ft. per sec., and the great water powers, like those at the Falls of St. Anthony, are even worse off than in a state of Nature; but this drawback is not so great as might be thought because the powers are able to utilize most of the storage when it comes during the period of navigation.

Such are some of the complications and drawbacks which are encountered in this reservoir system and which would surely be met in a system built up under less favorable natural conditions.

Nevertheless the Board found that the system was in itself a very great benefit and that the lack of appreciation of its advantages was for the most part due to ignorance of what they actually were. At the public hearing the opposition fell to pieces by the mere force of a better understanding, and it is safe to say that the system will

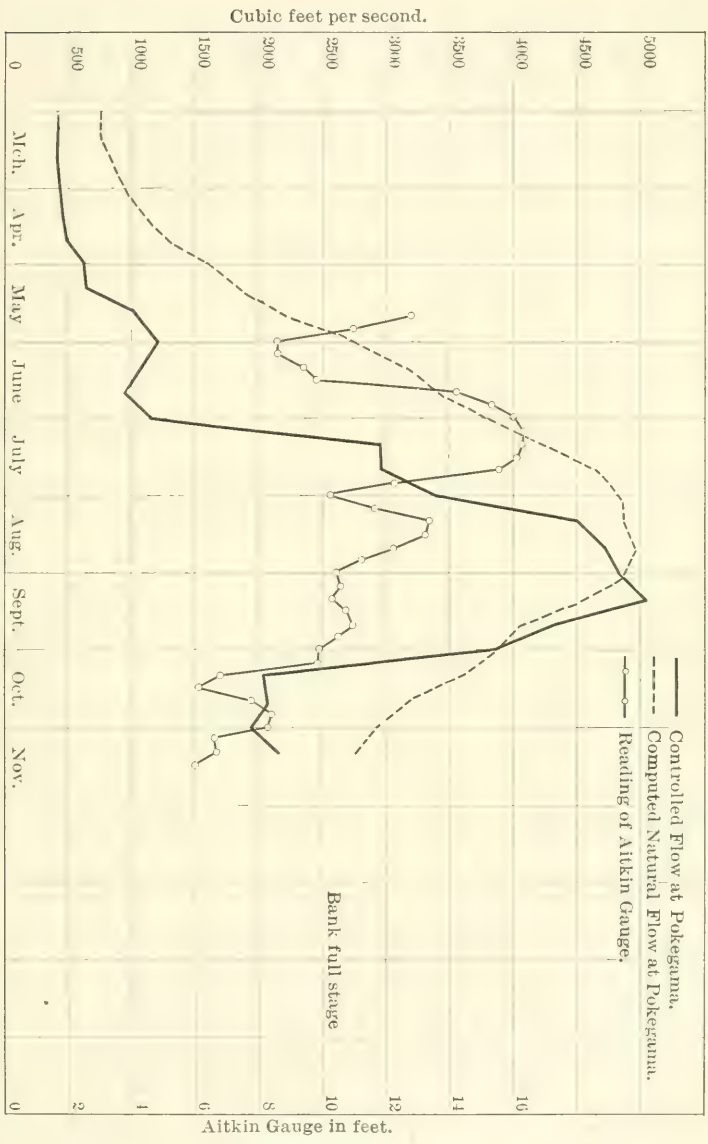


DIAGRAM SHOWING EFFECT OF UPPER MISSISSIPPI RESERVOIRS ON FLOOD SITUATION AT AITKIN, MINN., 1905.
Cubic feet per second.
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
Controlled Flow at Pokegama.
Computed Natural Flow at Pokegama.
Reading of Aitkin Gauge.
Bank full stage
16
14
12
10
8
6
4
2
0
Mch.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
1905
Aitkin Gauge in feet.

FIG. 2.

never be abandoned, but will be extended along the lines of the original project.*

The U. S. Geological Survey has recently proposed quite an extensive reservoir system for the Sacramento basin, similar in principle, though smaller in extent, to that of the proposed Ohio system. The flood problem of the Sacramento River is the most difficult in the United States in proportion to its magnitude. In fact it seems as if it will prove impossible to convey the extreme floods of that river to the sea without extensive overflow of the bottom lands along its course. The proposition to control the floods to some extent by means of reservoirs was elaborately set forth in the paper by Messrs. Clapp, Murphy, and Martin, previously referred to. The subject had already been considered by the Commission of Engineers appointed by the State of California in 1904 to devise a plan of flood relief. The Commission reported that, while any help from such a source must, of course, be welcome in solving the problem, it was very doubtful if such aid would be of sufficient importance to justify giving it much weight.† In discussing the paper above referred to, the author stated that, while he had never visited the sites in question, it was his opinion that, as to most of them, it would not be possible to realize over one-fourth to one-third of the benefits claimed, and he based his opinion on the published records of the flood of 1907, which was the greatest in the history of the river. George L. Dillman, M. Am. Soc. C. E., in discussing the paper, flatly pronounced the whole scheme impracticable and gave his reasons in detail for this conclusion.‡ Among them he cited in one case the great value of the lands to be flooded by the reservoirs, which he claimed were altogether more important for agriculture than for any diminution of flooding which the storage might cause in the valley below. In another case he cited the difficulty, which always suggests itself to an engineer in considering the subject, of timing the operations of the reservoirs so as to combine their effects to the best advantage, and particularly in keeping them empty in periods of prolonged precipitation, so that their capacity may be available at the critical moment. Other obstacles were pointed out, and the whole discussion presents another instance

*The report of this Board contains exhaustive data upon the system and its operation. It may be seen in the annual report of the Chief of Engineers for 1906, p. 1443 (Appendix AA, published separately in pamphlet form).

†Report, Commissioner of Public Works, State of California, for 1905.

‡*Proceedings*, Am. Soc. C. E., May, 1908, p. 464.

DIAGRAM SHOWING COMPARISON BETWEEN NATURAL AND CONTROLLED FLOW OF MISSISSIPPI RIVER BELOW THREE UPPER RESERVOIRS.

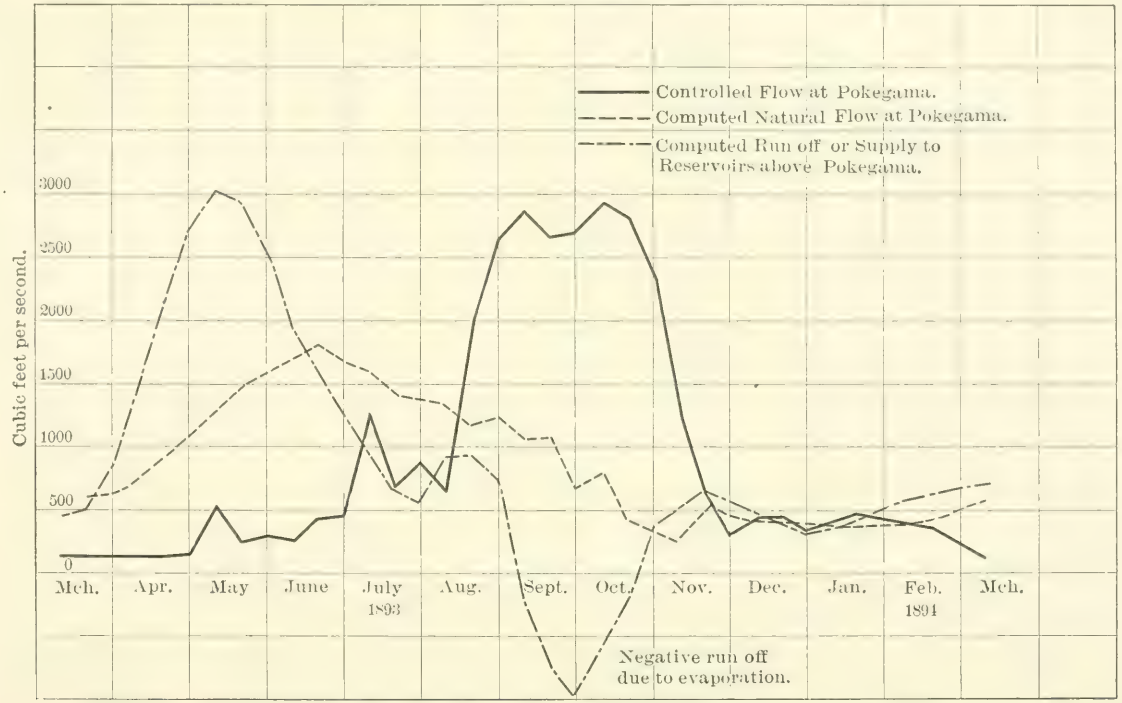


FIG. 3.

of the practical difficulties that stand in the way of any comprehensive reservoir scheme for controlling floods.

In 1903 the great flood of the Kaw River brought up the reservoir question again. Ex-Senator Burton, of Kansas, advocated the plan very urgently, stating in a speech at Kansas City that he "would have tens of thousands of reservoirs beginning at the headwaters of the stream and coming right down." A Board of Engineer Officers was appointed to investigate the practicability of providing against future disasters such as this flood had caused. The reservoir idea had made so deep an impression upon the public mind that a specific consideration of that feature of the problem was requested. In its report* the Board found adversely to the scheme on the ground that its great cost, conservatively estimated at $11 000 000, and the annual loss from the withdrawal of the necessary lands from occupancy, conservatively estimated at nearly $600 000, would not be justified on the ground of flood protection alone. Owing to the character of the country, this last consideration was particularly strong. The only real justification of so extensive a system in a country so largely devoted to agriculture would be its use in irrigation and power, and, if it became necessary for these purposes, doubtless a portion of it would be built.

The most elaborate study of this subject ever undertaken until very recently was made by the French Government, to determine whether reservoirs could be utilized to prevent the recurrence of such great disasters as the floods of 1856 in the valleys of the Rhone and other streams. A full résumé of these studies is given in the author's report already referred to on Reservoir Sites in the Arid Regions. The conclusion was the same that has been reached in every similar investigation. An interesting feature of the system then considered was that the reservoirs were to have sluices permanently open so that it would not be possible to close them entirely. They would operate, it was expected, to hold back a definite percentage of flood discharge—enough to keep the floods below the dams within safe limits. They would thus act automatically, just as forests are supposed to do. This was all right so far as the individual tributaries were concerned, but it was found, when the possible effect upon tributary combination in the main stream was considered, that, by holding back earlier portions of freshets and prolonging their run-

*Senate Doc. 160, 58th Cong., 2d Session, pp. 14-17.

off, they might actually swell the combination in the lower courses of the main stream.

Similar studies have frequently been made in all the principal countries of Europe, and in none of them, so far as the author is aware, has such a project on a large scale ever been undertaken or even favorably considered.

Coming now to the Ohio River, the immense importance of that stream as a factor in the floods of the Mississippi makes the regulation of its flow a matter of greater moment than that of any other stream. The project of controlling the run-off of its water-shed by means of reservoirs was urged very forcibly more than 60 years ago by Colonel Charles Ellet. The subject has often been considered since both in private and official investigations. The conclusion has invariably been that, great as the benefits of such a system would be if in existence, the cost of bringing it into existence would be out of all proportion to such benefits.

The scheme has recently been revived in a more attractive form, with data not hitherto available, and at a time when a period of heavy floods and much loss therefrom has turned public attention strongly upon the subject. Moreover, it comes supported by a comparatively new element in its favor—the vast expansion of water-power development made possible by the electric transmission of energy. The new presentation of the project is by M. O. Leighton, Assoc. M. Am. Soc. C. E., Chief Hydrographer, U. S. Geological Survey, and is understood to bear the approval of both the Interior and Agricultural Departments.* Mr. Leighton does not claim that his presentation is at all final or complete, but is rather a "statement of possibilities" which he believes are sufficiently promising to justify the Government in giving the scheme thorough investigation before further extensive steps are taken on present lines in the matter of flood control and channel improvement in the main rivers of the basin. Although an estimate of cost is submitted and certain conclusions are based thereon, it is stated that the data are too meager to give much confidence therein. Subject to these qualifications, the system, as set forth in Mr. Leighton's paper, embraces reservoirs on nearly all the tributaries

*The author has seen the description of the proposed system only as published in *Engineering News*, May 7th, 1908. He has had some correspondence with Mr. Leighton, and is under great obligation to him for a complete set of topographic sheets showing the various reservoir sites.

of the Ohio; the total cost is estimated at $125 000 000; the income from resulting water power at $20 per horse power, and a certain computed lowering of flood heights on the Ohio and Mississippi Rivers, and a corresponding increase in low stages, are given. The full details of the scheme are set forth in quite elaborate form. So far as the present criticism is concerned, the practicability of finding the necessary sites will be accepted, and only the estimate of costs and revenues and the deductions as to benefits will be called into question.

In their effect upon floods, admitting that all the reservoirs proposed can be built, the result must fall short of the claims put forth. If built at all, they must be built, as will be shown later, primarily for power development. It will never be possible, until Science can forecast the weather more perfectly than it is yet able to do, to regulate reservoirs for the maximum benefit of both purposes. This consideration is sometimes made light of, but nevertheless it is one of real importance. For industrial purposes the reservoirs should be full before the rainy season ends; for flood protection they should be so far empty that they may be able to hold back any flood-producing storm that is likely to come. While, doubtless, in a majority of years, a middle course could be pursued that would not involve much risk on the flood side of the question nor much loss on the power side, yet there would surely come exceptional seasons—the seasons of flood-producing rains or the seasons of great drought—when the reservoirs would be caught too full on the one hand or too empty on the other. Their full calculated capacity would not then be available for either purpose, and it is difficult to conclude that this would not happen frequently. In particular, if the reservoirs are really operated to prevent floods, it must often happen that dry weather will find them only partially filled, and that their full capacity will not be available either for power or navigation. This would not apply, of course, to a reservoir great enough to store all the run-off from its water-shed in the greatest known flood, unless considerable storage were left over from previous years—as is often done in the Upper Mississippi reservoirs. Mr. Leighton's estimates are based upon the mean discharge of the streams, which is, of course, greatly exceeded, possibly doubled, in very wet years. In any case it would seem to be necessary to hold ample capacity in the reservoirs as late as the end of March each year to provide for possible emergencies; but if

this is done there will be many years when the reservoirs will not fill.

An important consideration in the use of the reservoirs for flood control is that of a proper combination of their outflow. To any one who will try to figure out how this can be accomplished over a water-shed of such vast extent, with storms arriving at different times in the various portions, with no way of telling when, where, or with what intensity they will arrive, with the varying distances of the different reservoirs from those points where flood control is particularly important, the problem seems almost impossible—that is, impossible to realize the full effect based upon the aggregate capacity of the system. It is understood that Mr. Leighton has endeavored to do this, but it would be interesting to see the application to some of the great floods that might be designated. For example, in the flood of 1907, which reached its maximum at Cincinnati and Pittsburg about the same time, no amount of holding back of the storm water on the Upper Ohio at that time would have helped the situation at Cincinnati at all.

Another important consideration in the effect of these reservoirs, as they would have to be operated to prevent floods, is the great change that takes place in a flood wave as it propagates itself down stream. The author is unable to tell from Mr. Leighton's paper to what extent he has considered it. The paper itself seems to indicate that the discharge held back by a particular reservoir produces a corresponding volumetric effect (not gauge effect, of course) at all points below, after making a due allowance of time for the transmission of the wave. This would be an erroneous conclusion. For example, a wave that might rise at Pittsburg from 100 000 cu. ft. per sec. to 150 000 a day later and to 200 000 the next day, and then fall at a corresponding rate, would not at any point below produce a maximum increase of 100 000 sec-ft.; and the farther away the point considered, the less would be the increase. At Cairo, nearly 1 000 miles below, the same wave would take a much longer time in passing, probably not less than a week, and the maximum increase would probably not be more than 25 000 sec-ft. This is merely a general illustration, for exact data on the subject are not available. The problem is of such complexity that nothing but the results of long experience could establish a rule as to what might be expected in any given case; but

it can be stated with certainty that the diminution of discharge at any considerable distance below the reservoirs for a given time would never be as great as the amount held back by the reservoirs in the same length of time, and that, the quicker and the higher the flood, the smaller the relative effect at all points below. It is only when such wave elimination merges into a constant quantity, continuing for a considerable time, that the full effect of a reservoir would be experienced at any point below. This, in fact, is what would actually happen in the contrary case of the low-water season when the reservoir discharge is kept up for a long time.

Still another feature in the high-water effect of such reservoirs is the demand for water for power at all times. If there should ever result any really general use for all this water, as is predicted, then the consumption for power would make a considerable river in itself. Now this much cannot be shut off in any case. Street cars and shops must run and houses must be lighted whether the flood is ruining the lowlands or not. An example of this occurred in 1905 on the Upper Mississippi where the outflow from the upper dams was cut down to a minimum to reduce the flood in the valley at Aitkin, which was then being overflowed by the river. The mill at Grand Rapids, just below the reservoirs, made a strenuous protest and even threatened legal proceedings to compel the release of the full normal flow of the river.

Considering all the foregoing features of the operation of the proposed system, even if every reservoir were built with the full estimated capacity, it would be extremely fortunate if 75% of the predicted results, either in flood protection or in aid of navigation, could be realized.

It is in the matter of cost, however, that the weak point of Mr. Leighton's system appears. Judged by any reasonable standard, his estimates are hopelessly wide of the mark. The method itself of getting at a basis of cost is inadmissible. For example, in determining a unit of cost for that class of reservoirs which embrace the greater portion of the total storage, the figures for nine reservoirs are taken, counting as one the whole Upper Mississippi system. Only the Mississippi system has been built; two others are under construction and six are merely projected. In accordance with almost universal experience, and especially in view of the great advance in prices of

FIG. 1.—LEECH LAKE DAM.

FIG. 2.—LAKE CHEESMAN DAM.

all kinds since these estimates were prepared, it must be expected that these works, if ever built, will cost from 25 to 50% more than the estimates. Three of the projected dams are of the relatively cheap rock-fill construction, which would be inapplicable to most of the Ohio dams from considerations of safety.

The controlling element, however, in the unit estimate, is the Mississippi system, whose capacity is nearly one-third of the whole group considered and whose unit cost is only about one-seventh of the average cost of the others. The use of the Mississippi reservoirs in any way as a basis of estimate for the Ohio system is wholly inadmissible because of the dissimilarity of sites. The Ohio sites, with one exception, are dry sites—totally different from the lakes of Minnesota. Even the latter reservoirs could not now be built for three times what they have actually cost the Government. The flowage lands embraced about 80 000 acres, which were nearly all reserved while yet belonging to the Government. A few recent purchases of additional lands found necessary, and the experience now being met in acquiring the flowage rights for a reservoir at Gull Lake, show that, if these lands were to be bought to-day, they would cost from $10 to $25 per acre. The right of way alone would now cost twice as much as the dams.

Compare any one of these structures—Leech Lake, for example—with a representative masonry dam like the Cheesman Dam on the South Fork of the South Platte River above Denver, Colo. The author is familiar with both sites and once submitted a plan and estimate for a structure on the Cheesman site almost exactly like the one built. Lake Cheesman is a more favorable site than most of those on the Ohio system, for, although its capacity is not as great as some, the dam site is exceptionally advantageous, one of the most perfect in Nature—a very narrow gorge in solid granite, with a natural spillway already provided. In several of the Ohio sites, entire towns will have to be removed, important railroads will have to be re-located, a few mineral properties will be destroyed, and, in nearly all, road systems will be seriously disarranged. None of these conditions were encountered to anything like the same extent in the Cheesman site. Undoubtedly its unit cost, which is estimated at about $250 per 1 000 000 cu. ft., was as low as can be possibly realized on the Ohio system as a whole. Compare this with less than $5 for Leech Lake or $8 for the whole Mississippi system!

A recent example of projected storage is that presented by the late George Rafter, M. Am. Soc. C. E., for the Genesee River near Portage, N. Y. Owing to the moderate height of dam (apparently less than 150 ft.) and the large capacity of reservoir (15 billion cu. ft.), this is believed to compare favorably as to unit cost with the Ohio system. The estimate was $216 per 1 000 000 cu. ft. If it were to be built under the present conditions of the market, it would doubtless cost $250. It is understood that later investigations have shown that Mr. Rafter's estimate is only one-half large enough.

In 1895 the author made an extensive examination of storage possibilities in Ohio, near the divide between Lake Erie and the Ohio River, for the purpose of providing a water supply for certain projected canals. He prepared estimates for two sites on the head-waters of the Cuyahoga, for one site at the head of the Scioto, and for one at the head of the Great Miami. The estimates were based upon actual surveys and are given in detail in the report upon the subject.* The type of construction was not expensive. The total capacity was 11 000 million cu. ft. and the unit cost $300. To-day it would be at least $350.

Most of the proposed sites for the Ohio reservoirs are not advantageous sites. The topography of the country is unfavorable. The sites are not compact basins, like those occupied by lakes or ponds or mountain meadows, but are, for the most part, trunk valleys with numerous tributaries, nearly all of them quite narrow. They may be roughly compared to the form of the hand with the fingers outspread, the dam occupying the position of the wrist. The ends of the fingers are frequently many miles from each other and from the dam. Numerous villages occupy the valleys. The road systems of the local communities traverse them. The disadvantage that will result to public travel, by forcing it out of these natural routes over the hills and around the ends of the fingers, will be very great. The lands lying between the fingers, in some instances, will be so far cut off from convenient access that their value will be much impaired, and damages will have to be paid on that account. In several instances the necessary changes in railroad alignment in the hilly country will be extremely costly, if not impracticable. A great many cemeteries will have to be removed, which means, not only the cost of removal, but

*House Doc. 278, 54th Cong., 1st Sess., pp. 78, 83, 86.

extensive purchase of lands outside. Such drawbacks are, of course, encountered in all similar work, but they are excessive in these sites. They are mentioned solely from their relation to the question of cost. No one can examine the maps of these sites and not be convinced that the cost of right of way and damages alone will considerably exceed Mr. Leighton's estimate of the entire cost of the system.

An element affecting cost is that of safety. Owing to the situation of many of these proposed reservoirs, the results of failure of the dams would be so appalling that no chances can be taken. The structures can be made safe, of course (except against earthquakes), but it will cost money. Nothing short of the highest type of construction —masonry for all the larger dams—can be considered. Mr. Leighton has cited certain dams upon the integrity of which great interests depend as evidence of the confidence of engineers in these structures, but if he will apply their costs, particularly those of important structures in Europe, to his proposed system, the money value of safety will mount up to a prodigious figure.*

A feature of this question of safety often overlooked is the depreciation of the market value of property, due to its location below a dam where failure of the dam would mean a disaster of great magnitude. However safe the structure may be, many people would not purchase property below it, and its market would be correspondingly diminished. While such loss can hardly be made a subject for damages, it is a real loss to the owners.

These reservoirs being built for flood protection, the sluices must be very large, so that at times they can be discharged practically as fast as the water runs in. This will be necessary during periods of prolonged precipitation in order to keep the reservoirs from filling too full before the danger is past. This detail of construction will add largely to the cost.

*The recent failure of the Hauser Lake Dam on the Missouri River, near Helena, Mont., is a good illustration of how the unexpected may happen. Here was a dam built of steel and concrete, two materials whose properties are thoroughly understood. The case was one which "ordinary engineering" might be expected to handle successfully. The public had reason to feel confidence in the structure. Yet "it fell and great was the fall thereof," not only in the total wreckage of the dam, but in the losses caused along the valley below.

The accident affords also another illustration of the omnivorous claims put forward in these days in the supposed interests of forestry. The disaster was promptly cited as an example of the havoc wrought by floods in a country without forests. The normal flood discharge of the Missouri at this point is 20 000 cu. ft. per sec.; for 1907, it was 26 000 cu. ft.; the maximum on record is about 50 000 cu. ft. At the time of the accident the discharge was about 7 000 cu. ft. per sec.

Taking everything into consideration on the most liberal basis, it is evident that this system cannot be built for less than $250 per 1 000 000 cu. ft. The probable increase in the value of property to be condemned before the system could be built, and the present scale of prices of labor and material, make this figure a minimum. This would swell the cost of the whole system to over four times Mr. Leighton's estimate, or over half a billion dollars.*

This is not all, however. It appears that the complete development of the reservoir system as proposed will take from industrial use probably 1 500 000 acres of land, including the lands actually overflowed, the margins subject to damages, and sites for the dams and various structures appurtenant thereto. These lands will be in large part, by the very fact that they lie in valleys suitable for storage grounds, the best lands in the localities. Sooner or later they are bound to come into agricultural use, and with proper cultivation their annual net revenue value will be at least $5 per acre. If utilized for forest culture, they ought to yield 500 ft. B.M. of lumber and 1 cord of wood annually per acre. The value of the land for this purpose ought to be as great as the figures just given. It thus appears that the occupancy of these lands for reservoir purposes will take from the community an annual product of at least $7 500 000 worth, and probably more.

The reservoirs will store about 2 150 billion cu. ft. of water. Assume that this can all be utilized for water power, with the average head of 200 ft., giving theoretically about 1 600 000 h.p. per year, or 1 280 000 h.p. at 80% efficiency. At $5 per horse power (the basis for this figure will presently be considered), the revenue from water power will be $6 400 000, which falls short of the loss resulting from withholding the sites from productive use.†

Viewed in the light of the foregoing exposition, the weakness of the reservoir scheme, as a measure of flood control or for improving navigation, is at once apparent. The question is, Will the ends justify

*Recent examinations of certain sites, embracing nearly 70% of the proposed Monongahela storage, indicate that the whole Ohio system will cost at least a billion dollars, and possibly a billion and a half.

†The sanitary feature has not been considered, although it is one of some importance. The laying bare of large areas of reservoir bottoms in the heated portion of the year, is objectionable, but it is not a matter affecting the element of cost. Neither is much stress here laid upon the danger to the reservoirs from silting up. This is not a region of heavy silt movement. In most of the reservoirs, the process will be very slow, and we may safely leave to distant generations the task of dealing with this problem whenever it reaches an acute stage.

the means? If the ends sought could be attained in no other way, possibly they might; but they can be, and for a small fraction of the reservoir cost. Consider the estimate, already given, of $500 000 000. Take $40 000 000 and reinforce the entire levee system of the Mississippi. That will make it impregnable—as safe as any of the proposed reservoir dams. Take $60 000 000 and revet the banks of the Mississippi wherever necessary from Cairo to the Gulf.* The reservoir project does not touch this important matter at all. Devote whatever sum is necessary to the protection of the bottom lands of the Ohio basin. Give Cincinnati and Pittsburg each $10 000 000 to assist in local changes necessary for complete flood protection. Devote a sum to navigation such as our engineers have never dared dream of, and the Government will still save more than Mr. Leighton's estimate of the whole cost of the reservoir system. The more closely this reservoir proposition is scrutinized, as a scheme for flood prevention, the more impracticable it appears. It is only a trade-off at best. It is giving up to perpetual overflow valuable lands to save others from occasional and even rare overflow for short periods. Now if, at less cost, these lowlands can be better protected by other means, thus leaving both the valley lands and reservoir sites open to productive use, how much better it will be!

If the author were to venture a criticism on Mr. Leighton's attitude in this matter, it would be that he has not fully appreciated his responsibility in bringing forward again this old proposition without fuller consideration of its organic defects. This is well illustrated in the opening paragraph of his paper, in which he says:

"This report will be confined to a statement of possibilities. There will be no attempt to prescribe methods for treatment of each local modifying condition that will be encountered in the prosecution of the plan here proposed. Such features are merely collateral, and their proper disposition is a matter of ordinary engineering."

This is a complete reversal of his obligation in the matter. The "possibilities" of reservoir control have long been recognized. The logic of the plan is well understood. It has always appealed to the popular mind. In particular, reservoir control of the Ohio floods has been advocated for more than 60 years, and its possibilities have often been investigated. The plan has been uniformly rejected on

*Report, Mississippi River Commission, 1896, p. 3457.

one ground, viz., that as a scheme for flood control and navigation improvement its benefits would not justify its cost. It is, therefore, incumbent upon whoever revives the scheme to come well fortified upon this particular feature. He must give some study to the treatment of "local modifying conditions." It makes a difference whether he can go to a great natural lake like Winnibigoshish and store 40 billion cu. ft. of water for a mere trifle, or whether he must evict whole villages, disturb railroads and highways, absorb valuable lands and possibly subject communities to serious risk. These are the questions upon which the success or failure of the scheme depends. Yet Mr. Leighton brushes them aside, as it were, with a wave of the hand, as "merely collateral" features, matters of "ordinary engineering" only. Here is the weak point of his project. Weighed in the balance of practical accomplishment, either for flood control or navigation, it will be found utterly wanting, and the development of the system, as has always been held, will have to be based primarily and mainly on its value for industrial use. For the same reasons that the development of a great reservoir system in the Far West is justified by its industrial value—its use for irrigation—so a reservoir system for the Ohio, or any other rivers, except in a few unusual cases, must depend primarily upon its industrial value—the development of power.

In pursuing his criticism further, the author would not be understood to be "knocking," as current slang goes, the feature of the reservoir system just mentioned, because, in his judgment, there is no one thing in the present movement for the conservation of our natural resources that is more important than storing the flood waters of our streams for power development. It stands in the same category with the preservation and extension of our forests. It stands on even a surer basis, for Man, either wilfully or through neglect, can destroy the forests, but he can never diminish in the smallest degree the power of running water. It is a great solar engine, perennial and perpetual in its action. It requires no aid from Man in its production. All he has to do is to utilize it. Providentially electricity has unfolded its power to transmit this energy over great distances, and has thus made practicable a development which would otherwise have been impracticable. In time water power will replace coal and oil and will become the one great source of power, unless discoveries are made which are not now foreseen. The author thoroughly believes in devel-

oping this power through public agencies and preserving it from private ownership and control. His present criticism is directed not at all at the principle involved, but at the extravagant expectations now being fostered as to the possible revenue which the Government may derive from such development.

The quantity of power estimated in the publications of the Geological Survey and the Agricultural Department are based upon an assumption that most engineers will question, viz., that 90% of the fall of our rivers can be utilized in effective head upon water-wheels. This is too great a figure. The most thoroughly developed river in the United States, namely, the Merrimac in New Hampshire and Massachusetts, develops only 70% of the total head. Taking all the streams into consideration, it seems hardly possible that more than 50% of the fall can be utilized. When the fall of a river is uniform, even if quite steep, the cost of long canals or high dams necessary to concentrate it at one point often prohibits development altogether. From altitudes of 3 000 ft., the Missouri and Yellowstone, for example, descend to the sea with a total energy of possibly 5 000 000 h.p.; yet comparatively little of this can be developed advantageously. It is only in those places where Nature has helped out by concentrating the fall at cataracts or rapids that water-power development is commercially profitable. At low dams, such as are ordinarily built at lock sites, the head is often nearly all obliterated during high water. How far storage may affect these drawbacks cannot be said, but it should, of course, help a great deal. The official estimates of flow for non-regulated streams are based on two weeks' average lowest flow. This may probably be extended materially with reservoir aid or supplementary steam power. Possibly the total estimated horse-power may ultimately be realized.*

When it comes to the royalty which the Government may receive for these water powers, if developed by private interests, the price of $20 per horse power, adopted by the Geological Survey and the Agricultural Department, is wholly out of the question under present conditions. Possibly the author does not understand what the figure is intended to embrace. From Mr. Leighton's articles, the inference has been

*There has recently been invented a device called a fall increaser, an adaptation of the Venturi meter, by Clemens Herschel, M. Am. Soc. C. E., which promises to utilize the extra flow of streams in time of flood water and low heads to increase and maintain the head upon the wheels. If this invention proves a success, as seems probable, it will be an immense gain to all water powers of low head subject to large fluctuation, as would doubtless be the case in very many of those under consideration.

drawn that wherever the work of the Government renders power available which was not available before, either by building dams, as at lock sites, and thus creating a head, or by storing water which might supply powers below with more than they would have without, the value of the power thus rendered available should return to the Government $20 per horse power per annum—an "exceedingly low price," as Mr. Leighton puts it.*

It is not understood that the Government is to build the power plants, but that this is to be done by the interests availing themselves of the privilege. Estimates of undeveloped water powers on many streams of the Atlantic slope, by the Geological Survey, leave one to infer that these powers are considered worth at least $20 per horse power to the Government even without dams or reservoir aid. While the statements are not clear as to what is actually meant, the various references to resources to be derived by the Government from these powers lead to the above conclusion. It would be of advantage in considering questions involving these published estimates, if the basis for this $20 price or royalty could be made more specific.

Under present conditions, or such as can be reasonably foreseen, no such royalty is possible except in extraordinarily favorable circumstances. Efforts which have been made to derive a satisfactory revenue from existing powers do not justify any such prospect. The many and various practical difficulties in exploiting these powers are rarely appreciated by those who have not encountered them in actual experience. The cost of water-power development is restricted to narrow limits, if it is to compete with coal. An engineer of high standing whose life-work has been connected with water-power development says:

"I am advised that, with good coal at $2.00 per ton in this territory, the cost of fuel per horse power per annum (300 days of 24 hours each) is less than $8.00 for producer gas engines and for steam power about $12.50 in large size equipments. In many localities coal will cost even less than $2.00 per ton, allowing thus a still wider margin. If we now consider the usual and unavoidable handicaps and incumbrances to all water-power installations, such as floods, low water, ice-flow, back water, etc., etc., we have conditions which will make it a serious study for any power consumer to determine if the balance is not considerably against water power in that particular territory, at this time, from a

*"On the Youghiogheny alone, where it is proposed to install a slackwater system comprising three locks and dams, at an expense of $600 000, proper development of storage will insure the production of a minimum of 4 100 h. p., the value of which, reckoned on the exceedingly low price of $20 per horsepower year, would produce a total income of $82 000."

purely commercial standpoint. At any rate it must be obvious that no such rate as $20 per annum per horse power can be paid to the Government by any power user for the right to draw the water only, and besides this, stand the expense of installing and operating the water-plant."

Another hydraulic engineer of national reputation says:

"I think that as a general proposition, the suggestion that all water powers to which the Government consents should pay royalties, and especially where the parties own their riparian rights, would tend to defeat the development of most water powers and would certainly very much curtail the number of water-power developments. I am impressed with these conclusions because of the present difficulties in financing good water-power propositions."

In *Power*, May 19th, 1908, is an article by Henry Docker Jackson, in which a critical comparison is made between steam and water power. In this article occur the following tabulated estimates of cost of installation and of annual operation, based upon a (theoretical) installation of 1 000 h.p. The costs are averages of a number of different plants:

"PLANT COST.

Plant.	Steam.	Water.
Building and works	$10 000	$77 000
Engines, boiler, etc.	48 000	.....
Turbines and generators	15 000	17 000
Trans. lines, etc., 20 miles	.....	40 000
	$73 000	$134 000

"FIXED CHARGES.

Interest	$3 650	$6 700
Fuel, $2.50 per ton	6 160	.....
Water	600	.....
Operation	3 400	3 800
Oil, waste, etc.	770	500
Maintenance	1 500	2 680
Depreciation	3 650	3 500
Taxes	1 460	2 680
Insurance	650	.....
	$21 840	$19 860
Cost per I.H.P. per year	$21.84	$19.86

"COST PER HORSE-POWER-YEAR.

	Steam, 88%.	Water, 95%.
100% Load factor	$24.82	$20.90
75% Load factor	39.92	32.00
50% Load factor	54.60	45.00"

From the last part of these tables, it is very evident that a royalty of $20 per horse power would turn the scale wholly in favor of steam under all conditions of load. In fact, it is reasonably certain that $5 per horse power per annum would be an outside figure, and even this would often be prohibitory. The situation will not necessarily be improved by the growing demand for power, but rather by the diminishing supply and increased cost of fuel. So long as coal can be had for anything like present rates, no very great charge can be realized from water power wherever fuel is readily available. Under present conditions $120 per horse power may be considered as an average limit for first cost of a water-power plant, if it is to compete with steam. A charge of $20 per horse power per annum would be equivalent to doubling this first cost.*

A variable element in the cost of water-power development is the distance from plant to market, or the length of the transmission line. When this is very great, as in numerous plants in the mountain districts of the West, it makes a large addition to cost of installation and must correspondingly reduce the royalty that could be paid for the power itself.

An interesting example of what the Forestry Service has been able to do in this line with unimproved water powers is that of a recent permit for the development of a large power in the Cascade Mountains within the forest reserve. The beneficiary of the privilege is required to pay annually for "conservation," 10 cents per 1 000 kw-hr.—equivalent to 65 cents per horse power per year continuous running. The right is retained by the Government to increase this charge 25% every 5 years for a period of 40 years, after which the whole arrangement may be readjusted. The maximum charge at the end of the 40 years will, therefore, not exceed $4 per horse power.

The only way in which a rental of $20 per horse power can be obtained with any degree of certainty, and that in only a small proportion of the localities for many years to come, is for the Government to build the plants. It is admitted that this suggestion will grate harshly on many ears because of its newness and its departure from

*Mr. Leighton cites the Falls of the Ohio as an example of an opportunity to develop 110 000 h. p. by aid of his proposed reservoir regulation. This, he states, at $20 per horse power is 2% interest on $73 000 000. To any one familiar with the physical conditions involved in the development of this power, it will appear extremely doubtful if any company could guarantee to deliver continuously this amount of power, even with the full aid of reservoir regulation, and pay any royalty whatever.

the established ideas. But a little consideration will show it to be not only the best way for both private and public interests, but really the only practicable way. This may be illustrated by a concrete example:

The Government has just completed a survey and adopted a project for the construction of what is known as the Lake Washington Canal in the City of Seattle. It is a canal to connect Lakes Union and Washington with Puget Sound. The discharge from the tributary water-shed which will flow through the canal averages about 1 500 cu. ft. per sec. The mean fall at the lock site is about 15 ft. The theoretical energy is about 2 500 h.p., but owing to the tidal fluctuation and variations of flow with the seasons (which cannot be wholly eliminated on account of the necessity of limiting fluctuations of level in the lakes to about 3 ft.), and also to the requirements for canal power, lockage and leakage, it was thought that about only 1 000 h.p. could be depended upon with certainty for outside use. As this power is located in the heart of a great city, it seemed as if it ought to be turned to good account in helping bear the cost of maintaining the canal. Efforts to obtain tentative propositions for developing this power were, however, wholly fruitless. The plan was then considered of having the Government build the plant and lease it to consumers of power. On this basis a tentative offer was obtained from a responsible consumer to take the plant, operate it, keep up all repairs and pay the Government $18 per horse-power year. Probably by the time the canal is completed, a figure of $25 can be obtained, and as more than 1 000 h.p. will probably be developed, it is likely that the Government will receive upward of $30 000 per year for this power—enough to pay the entire cost of operating the canal. The extra cost to the project of adopting the power-plant feature is $220 000, so that the revenue will be nearly 14% upon the expenditure.

In recommending this plan to the Department, it was pointed out that the true advantage of the Government, even apart from the revenue expected, favored its adoption. It simplified the whole relation between the Government and the consumer. If private interests were to build the plant, they would acquire vested rights which would always stand in the way of future control and lead to complications if it should become necessary to terminate the arrangement. With the plant in the possession of the Government and the

users standing simply in the relation of lessees for a limited period, without great initial expense on their part, and with freedom on the part of the Government to control the arrangement without the complication of private ownership, the whole plan would stand on a simple, practical, business basis. This view prevailed with the Department and is now before Congress for adoption, being possibly a departure in this line.

The principle involved in this case should be given general application. In addition to avoiding complications with private ownership, there are other important considerations. When a power is developed or a reservoir built, it should be so planned from the start as to bring out its full possibilities. A private company can rarely do this. Generally its scheme does not require it, nor its resources permit; but a site once occupied by an inferior work may be perpetually barred from complete development. Moreover, in any such work, the Government can derive a greater benefit than any private individual or association. A private company must build for the immediate future; it cannot wait long for dividends and it can generally realize only on such application of the power as is possible in the immediate vicinity. The Government, on the other hand, derives all the benefits which come from the stored water anywhere on its course from the reservoir to sea. These benefits arise from all the powers through which the water flows; from the improvement of navigation and the prevention of floods and from every other use to which the water can be put. Furthermore, the Government is building for all time, while the individual builds only for the present and near future. The case is similar to that of landlord and tenant. A tenant cannot afford to make improvements on the farm because it is not his and he may remain on it only a short time. The most he can do is to get out of the farm what he can in its actual condition. The owner, on the other hand, can put in improvements which yield him no immediate return because he holds the property long enough to realize upon them. So it is with the Government; it can wait for realization upon its improvements much longer than a private company. In forestry, for example, no individual can afford to wait from three to ten generations for a crop. Only the Government or a great railroad corporation can do this. Likewise, in building great reservoirs, no private company can build for the distant future. It

is only the landlord that can make such far-reaching improvements upon his estate.

Wherever, therefore, there arises any real demand for power development at the site of any Government work, as a lock and dam, the judicious course would seem to be for the Government to prepare a comprehensive plan for development capable of being carried out progressively as the market for power may justify. Let it then build the plant as fast as needed and lease it to private agencies under suitable restrictions. Likewise, when the building of a reservoir promises to be of obvious utility, and the conditions are such as to make it properly a subject of Government adoption, let the Government build it, utilizing the water in its own plants below and collecting a revenue from private plants that may use it. Whenever at the time of construction there is a direct return in sight of 2 or 3%, it should be considered justifiable from a Government point of view. The certain enhancement in the future value of such utilities and the incidental advantages in flood protection and navigation make this a conservative proposition.

That difficulties will be encountered in deriving the full return from its work to which the Government would be entitled cannot be denied. This would be the case particularly wherever it is a question of compelling existing power plants to pay for the extra water they might receive through Government storage. This question came up before the Mississippi Reservoir Board in regard to the powers at St. Anthony's Falls which derive such benefit from the reservoirs. The Board remarked as follows on the subject:

"It may be urged that if the incidental benefits of the reservoirs to the water-power interests are so great, these interests should be required to contribute something to the maintenance of the system. There would doubtless be a willingness to do this if a satisfactory method could be found. But there is no practicable method of enforcing any charge upon the use of this water. Where water is taken in a separate channel from above a dam or lock and conducted to a mill, it is a simple thing to measure it and to cut it off if it is not paid for. But when it must be let into a natural stream, where it mingles with the run-off from below, it is impossible to determine what proportion of stored water the mill may be using, or to enforce its non-use if not paid for. But, if such an arrangement is not practicable, that fact does not constitute an argument against the reservoir system. So long as the reservoirs are performing the service for which they were

created, every additional benefit derived from them is only an additional argument in their favor."

These disadvantages will adjust themselves in time.

Such, in the opinion of the author, must be the basis of any great reservoir system in our country—industrial use. Even in the uniquely favorable conditions at the headwaters of the Mississippi, no one can doubt that the real purpose being served is that of mill power, whatever the theory upon which the reservoirs were built. The great system of the Far West is being built for irrigation, power and domestic supply. So on the Ohio and other eastern streams, the system must rest upon an industrial basis and expand only as industrial demands justify. The innovation involved in building reservoirs with public funds for these uses is admitted; but it is no greater than it was ten years ago to build them for irrigation. When the author was investigating that subject in 1896-97, he found a widespread opposition throughout the arid regions against Government control of irrigation works in any way, and in his report he went no further than to advise the building of reservoirs for giving the people more water, leaving its distribution exactly as it was before. Yet in the short space of ten years public sentiment has completely changed and, to-day, no one questions the wisdom of the broader plan upon which these works are being carried out. So it will surely be in regard to reservoirs in all other parts of the country. The principle is the same. It may be accepted that only the General Government can do this work in the comprehensive way in which it ought to be done, because only the Government can reap all the benefits; only the Government can wait the long periods necessary for full returns, and only the Government has the necessary resources to make expenditures on the required scale. These points will not be enlarged upon, and the many and cogent reasons why this is so will not be given. The trend of public thought is all in that direction. The old idea that the Government cannot execute great works or small as cheaply, efficiently and expeditiously as private agencies is fast being dispelled, and the vast benefits which the people derive from public control of important enterprises are coming into fuller recognition all the time.

The foregoing remarks should not be construed as in any way rejecting the idea of local help by states, counties, cities, or even private

agencies. It often happens that public works have a special local importance in addition to their public value. It is just and proper in such cases that local aid be given. This principle is now fully incorporated in river and harbor legislation. For example, the Lake Washington Canal, which will be of very great local importance to the City of Seattle, is a joint enterprise between the Government and the City, the latter paying fully one-third of the cost. The co-operation between the U. S. Geological Survey and the several States in preparing a contour map of the country is an example on a large scale. The principle ought to find an extensive application in the establishment of national forests throughout the country.

Conclusion.

This paper will be closed with some reference to the relation of navigation to other uses of our streams, and to certain legal obstacles that stand in the way of comprehensive measures. That the improvement of our inland waterways should be organized upon a more rational system than it has ever been, that the reciprocal relation between navigation, water power, etc., should be given practical recognition, above all, that the prosecution of these works should be placed upon the same sure basis as is the construction of the Panama Canal, with positive assurance that, when once commenced, funds will be forthcoming for their prompt completion, would seem to admit of no doubt. How far navigation should be correlated, in improvement work, with other uses of the streams, is an open question. Water power and navigation are in many cases so closely related that they will have to be considered together. In regard to soil wash, no such intimate relation exists. To whatever extent soil erosion now exceeds that of former times, it relates almost exclusively to cultivation and has no appreciable influence upon the channels. Its control is of far greater importance to agriculture than it is to navigation. This is also true of irrigation, which, so far as it affects navigation at all, affects it injuriously. If the development of irrigation is ever carried to the length that we hope it may be, it will cause a heavy drain upon the low-water flow of the Missouri, Sacramento, San Joaquin and the Columbia Rivers (not important as to this stream), the only navigable waterways of consequence that are affected by it. Except for this fact of drawing water from the streams, irrigation has no relation to navigation.

Forestry, irrigation and prevention of soil wash are all related to the conservation of the vegetable resources of the country. They are kindred purposes and should naturally fall under the same administrative control. Navigation is a function of transportation, which is a very different subject. Water power is becoming more and more closely related to it, and these two subjects naturally go together. It must not be expected that the character of works for river regulation can be materially changed by means of reservoirs, forests or soil-wash prevention. Levees and bank protection, locks and dams, dikes and dredging will continue to be standard methods of river improvement in the future as in the past. The accumulated experience of centuries in all civilized countries cannot be set aside in a moment. In particular, flood protection is not likely ever to find any complete substitute for levees. They have been used extensively the world over throughout recorded history. People who think only of the Mississippi and the Po, when levees are mentioned, little understand to what an extent "diking" is resorted to wherever rich bottom lands have to be guarded against floods or tides. Some of the finest agricultural lands in the world are behind levees where almost perfect security is felt. No class of river control is in more extensive use, none is better understood, and from none has the world, throughout its history, derived greater security and benefit.

Municipalities, like Pittsburg, Cincinnati and Kansas City, must look in the main to their own efforts for protection against floods. In particular they must reject absolutely the delusive promises of forestry. These cities are trespassers upon grounds dedicated by Nature to a condition of overflow. They have occupied these grounds and placed themselves in the way of the floods deliberately and with their eyes open. They have gone farther than this, and in many instances have encroached upon the channels and have thus made the floods worse than they used to be. It is not for them now to look for outside deliverance, but they themselves should grapple courageously with the problem. In most cases these problems admit, if not of complete solution, at least of a very large measure of relief. The maxim that Providence helps them who help themselves may also apply to the Government. Co-operation in connection with its regular work, either in channel improvement or in the building of reservoirs, would doubtless be given. The disposition which must be met and

overcome is to let things go as they are, trusting blindly to chance to deal more kindly in the future. This supineness of spirit and the enervating reliance upon indefinite future relief through the agency of the Government must be replaced by self-reliance, and these great industrial centers must rise in their own might and free themselves from their bondage to these ever-recurring catastrophies. In Boston, Chicago, Galveston, San Francisco, and even that lusty young giant of the Northwest, Seattle, are examples enough of what an aroused civic spirit can do in the direction of self-aid.*

The part that reservoirs will play in the larger problems of channel improvement and flood control on the great rivers will be in the nature of an insurance. Every cubic foot of water taken from the crest of a flood and released when the rivers are lowest is *pro tanto* a benefit. If the great floods of the Mississippi can be cut down by so much as a foot through reservoir storage, it will be an immense gain; and the same will be true if the low-water stages can be increased by 2 or 3 ft. Whether the much greater results expected by Mr. Leighton can ever be realized is a question which the future alone can determine.

A word, finally, concerning the legal obstacles in the way of a broad Government policy looking to the development of National forests and the storage of water on an extensive scale. The expansion of Government work into fields of obvious utility is often blocked by the structure of our Government through the bar of constitutional prohibition or at least lack of power. It is said that the purchase of lands for the rearing of forests for timber alone is unconstitutional, and that the same is true of the storage of water for any other purpose than navigation; and yet, forests for timber and reservoirs for power must always remain the real justification for public expenditure along these lines. To the average understanding the distinction between things constitutional and things unconstitutional is often hard

*The author is not closely familiar with the situation at Pittsburg and Cincinnati, but he is familiar with that at the two Kansas Citys where, in 1903, the greatest loss occurred that any American city ever sustained at the hands of a river flood. He speaks from the results of careful study on the ground when he states with the utmost positiveness that, for approximately $10 000 000, with such aid as might reasonably be expected from the Government on the Missouri River front, the flood problem of the Kaw and Missouri in that hive of industrial enterprise known as the West Bottoms can be solved absolutely; the too small area of these bottoms can be increased by upward of 200 acres; two-thirds of the bridges in the same area can be eliminated; that prodigious barrier to free movement—the Kaw River—can be practically removed or placed where it will not be in the way, and the general situation can be so improved that the resulting benefits, wholly apart from that of flood protection, would be well worth the cost.

to discern. The Government is now expending millions in storing water and conducting it upon land whereby the products of the soil may be obtained. It is applying this water to both public and private land, or to lands that were in private ownership when the projects began. Is there any real difference between providing the power to raise sugar beets, for instance, and that for manufacturing them into form for human consumption and transporting them to the consumer? Are not the last-mentioned purposes quite as necessary as the first?* And again, is there any distinction in principle between improving a river so that boats can navigate it and improving it so that it may provide power that will transport produce by land as well as by water?

Again, the Government has accepted gifts of land like the Yosemite Valley and the Muir Redwood Grove, to be given over to the enjoyment of the people and involving perpetual expenditures for maintenance in the future. It has traded lands of its own for lands with which it has parted ownership. It reserves vast areas to-day which might be private lands to-morrow. What is the distinction of principle between doing all these things and buying outright lands that are needed for the same or similar purposes? They are distinctions without real differences. They concern the letter and not the spirit, and they cannot stand whenever the interests of the public really demand their abrogation.

Still, it is probably a fact that Federal authority to buy lands for forest culture alone and to create reservoirs for industrial use exclusively, would be considered by the Courts as transcending the power of Congress under the Constitution, and it is this fact that forces those who believe in having the Government do these things to strain the truth by attempting to prove that they are necessary for navigation and for the prevention of floods. It enforces a policy of indirection instead of permitting these things to be done squarely for their real purpose and as a matter of right. In his address before the Judiciary Committee, in its hearing on the Appalachian Bill, Mr. Pinchot stated that that proposition must stand or fall upon the theory that the forests regulate stream flow, and are therefore useful to navigation. Did he not refer to the particular point here under

*It has even been hinted by high judicial authority that the Reclamation Act itself would not stand the test of constitutionality, if brought into Court.

consideration—that on any other theory the measure would be unconstitutional? Surely he did not mean that the cause of forestry itself must stand or fall upon any such issue.

Does not this situation suggest the necessity for an important initial step which shall sweep away these artificial barriers and let these great questions stand or fall on their intrinsic merit? If the upbuilding of new forests, if the storing of our flood waters, are necessary measures for the welfare of the Nation, the way should be cleared for their accomplishment. There may be differences of opinion about amending the Constitution in the interest of uniform divorce laws, popular election of Senators and the like, but, if we may judge from the universal agreement upon the particular subjects here considered, every State in the Union would ratify an amendment giving to Congress the power to legislate for the conservation and development of the natural resources of the country.

The author should possibly state, in justice to the official body of engineers to which he belongs, that the arguments presented in the foregoing paper are his individual opinions only. He is not acquainted with the views of any other officer upon the subjects treated, except as he has seen them expressed in official reports or in the public press.

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THE MAINTENANCE OF MACADAM AND OTHER ROADS.

An Informal Discussion at the Annual Convention, June 25th, 1908.*

By Ira O. Baker, M. Am. Soc. C. E.

Mr. Baker. Ira O. Baker, M. Am. Soc. C. E.—In many of the States of the Mississippi Valley there are large tracts of rich alluvial soil almost entirely devoid of either gravel or stone suitable for building wagon roads. Many of these tracts are at present in a high state of cultivation, and all of them promise to be comparatively soon. Some of these tracts are probably as rich and productive as any equal area in the world; and the increasing population of the country and the intensity of the scientific study now being bestowed on agricultural operations in the United States promise speedily to increase still further the productiveness of the soil and thus add to the wealth of these communities.

The problem of securing and maintaining suitable wagon roads in these districts is not a simple one, either in its political or its material aspects. There is great diversity of opinion as to what constitutes a road suitable for the needs of these communities, some contending that the earth road meets all the necessary requirements, and others that the gravel or macadam road is an economic and social necessity. It is the purpose of this discussion to call attention to some of the difficulties in the case, although the questions involved are political, social, and economical, rather than engineering.

Before considering the subject positively, it may be wise to look at it negatively; that is to say, before trying to discuss the problem

*Continued from August, 1908, *Proceedings*.

one should get clearly in mind what it is not. During the 10-year period beginning about 20 years ago a large amount of literature upon the subject of rural road improvement was set afloat in newspapers, magazines, and reports, and, because of this flood, the wagon roads of the country districts have made less progress in the last 15 or 20 years than any other phase of rural life. Mr. Baker.

Facts and Theories Inapplicable.

About 20 years ago, two or three well-organized manufacturing interests, in order to increase the sale of their goods, deliberately started an agitation for better roads, and, unfortunately, committed the conduct of the scheme to those who knew little or nothing about either road construction or the larger principles of economics and political action involved in any comprehensive system of road improvement. The interests back of this propaganda were very skilful in arousing the enthusiasm of their followers, who were both numerous and widely distributed, and an attempt was made to carry by storm a reform system of constructing and maintaining the wagon roads of the nation. Many politicians and newspapers sought to secure popularity by favoring the proposed reformation, and, as a consequence, wide circulation was given to certain facts and theories, which in their proper field were true, but which when presented without limitation were wholly false and entirely misleading.

These intemperate advocates of road reform frequently cited the wagon roads of England, France, and Switzerland as examples of what the United States could and should do, regardless of the fact that those countries are abundantly supplied with good road-building materials, while in this country there are areas, greater than the combined area of those countries, in which there is absolutely no road-building material. Again, the stone wagon roads in those countries were built under the stimulus of military necessity and commercial need, before the advent of steam railroads; while in the greater part of this country the railroads have been the pioneers, and now there is no commercial need of long lines of wagon transportation, except perhaps in the immediate vicinity of large cities. Further, the density of population, the industrial occupation, and the agricultural methods in those countries are very different from the methods prevailing in this. Still again, the method of maintaining wagon roads in those countries, with their dense and poorly-paid population, is no criterion by which to judge what is wise or possible in this country. And once more, the political and social ideals on the two sides of the Atlantic are very different, and make possible certain results in Europe which are impossible in the United States.

Again, these enthusiasts frequently cite the experiences of local cities on Long Island and in New Jersey, within a few miles of

Mr. Baker. New York City, as though they were representative of rural conditions in general. New York is the largest and wealthiest city on the North American continent, and it is not to be wondered at that the building of good roads is one means of enticing people to move from the densely populated districts of the city into the more healthful suburban regions, and that the influx of such inhabitants has greatly increased the value of real estate; but not all of the United States is situated in the bedroom of New York City. Some years ago the annual report of the State Highway Commissioners of New Jersey contained an interesting account of road improvement in Gloucester County, which has been very widely copied and commended; but when it is known that this county is just across the river from Philadelphia, and that the soil is very sandy and the road improvement simply enabled truck farmers to wagon their produce to market, it is seen that this example has but little practical bearing on the general question of rural road improvement. Again, along the eastern shore of New Jersey there is a continuous line of summer resorts, where people of wealth go to enjoy themselves, and it is doubtless a good business proposition for such communities to build first-class stone roads for the pleasure of their profitable guests; but their success does not prove that a truly rural community should follow their example and also build expensive roads. The examples of the States of Massachusetts and New York in hard-road building are frequently commended to the inhabitants of the Prairie States of the Mississippi Valley, but the industrial and topographic conditions of these two States are not guides for the flat, grain-raising States. The primary fault of most road reformers has been that, through dense ignorance, superficial knowledge, or deliberate intention, they have presented facts, separate and apart from their limitive conditions, in such a way that the truth becomes a falsehood.

This ill-advised and intemperate road agitation has in many cases done harm in the communities most needing road improvement, since a form of road construction has been advocated which, under most conditions, has been entirely impracticable, if not ridiculous; and the result has been that those who are most interested in good roads have been most concerned to prevent road construction and maintenance being made subservient to interests foreign to those who use the roads most and also pay for them; and, as a result of this condition of affairs, the improvement of local roads has not received the attention recently that its importance warrants, and has not kept pace during the past 20 years with rural development in other lines.

Furthermore, all these road enthusiasts claimed that the only good road was a hard road; they were also very vehement in the assertion that nothing could be done to improve the earth road. In many localities a hard road was politically and economically impossible, and

hence the earth road was the only form available; but the assertions of the good-road enthusiasts only serve to fix the belief that nothing could be done to improve the roads of such localities. Mr. Baker.

Fallacies in Road Economics.

It was in the field of economics that these road reformers made their most glaring errors, produced the most irritation, and aroused the most antagonism. In this field the farmer was most capable of independent judgment. He was not much concerned about the lessons which the road enthusiast sought to draw from European experience or even from experience along the Atlantic Coast, for the farmer knew that social and industrial conditions in those localities were very different from those in the Mississippi Valley, and hence such arguments made little or no impression on him; but when the hard-road enthusiast began to tell the farmer how much it cost him to haul his produce to market, and how much he could save by the construction of hard roads, he knew instinctively that the conclusions were ridiculous, and the continual harping upon these false statistics and absurd estimates led him to believe that an attempt was being made to force hard roads upon him, whether or no, and his attitude changed from one of indifference to one of open hostility to all road improvement.

A brief examination of a few of the claims of these agitators will be made in order to see whether or not the farmer was justified in his judgment as to their truthfulness. It would not be worth while to refer to this literature except that by so doing its nature may be exposed and thus prevent it from being used in similar discussions in the future. The speaker will examine three articles—of the most reputable parentage—which have been most widely circulated in their original form, and are most frequently quoted. These articles seem to have been distributed to all the public libraries, and, apparently, when any one is moved to prepare a speech on road improvement or write an article on that subject to be used as plate matter in the country newspapers, he consults such literature. The speaker has seen evidence that each of the articles about to be referred to was used recently in connection with the meetings called to secure a good-roads plank in the platforms of the two principal political parties.

First.—One of these articles is a 64-page pamphlet entitled: "The Gospel of Good Roads, A Letter to the American Farmer," published by The League of American Wheelmen. The author says:

"From official Government sources I find that the farmers of this country, in the year 1890, had upon their farms draft animals as follows:

"Kind.	"Number.	"Value.
Horses	14 213 837	$978 516 562
Mules	2 331 027	182 394 099
Oxen	36 849 024	560 625 137
Total	53 849 888	$1 721 535 798

Mr. Baker. "You see you have nearly $2 000 000 000 invested in motive power of a perishable, uncertain, and expensive kind. Busy or idle, these animals must be fed and cared for every day. They are boarders that you can't get rid of when the busy season is over, and it stands you in hand to keep them at work."

The author of the above had surely not traveled much, or he would have learned that there were not anything like two and one-quarter times as many oxen in this country as horses and mules. Evidently, where the census report said cattle, he read oxen! The man refers to a fellow hard-road enthusiast who "estimates" that "bad roads cost the farmer $15 per year for each horse and mule;" and calculates that this loss amounts to $250 000 000 per annum. He next asserts, apparently offhand, that the loss due to "wear and tear on wagons and harness is $100 000 000." He then proceeds to add together the value of his draft animals, the alleged cost of bad roads, and the estimated cost of wear and tear on wagons and harness, and gets a total of $2 350 000 000. The sum is frequently quoted as the annual cost of bad roads in this country.

The above computations are almost as good as anything in *Puck*. Were not some of the horses and mules utilized in the cities? Think of the ludicrousness of including the value of each horse and mule in the cities (including those then used on the street railroads) plus $15 in determining the cost to the farmer of bad roads. Isn't it cruelty to animals to make the colt in the farmer's pasture bear the burden of a draft animal; or to charge the maverick on the plains with being responsible for part of the cost of bad roads? The wear and tear on wagons and harness had already been included in the preceding $250 000 000, the cost of bad roads; but an extra $100 000 000 is nothing to a hard-road enthusiast. Of course it is a small matter to have added the value of the draft animal to the alleged difference of its earning power on good roads over that on bad roads! The article makes an error of $2 100 000 in determining $250 000 or less! But such an error is of no moment to the road reformer!

There are several other things in this pamphlet almost as misleading, but the foregoing item is referred to here because it is the part most frequently quoted.

The pamphlet is liberally illustrated by cuts made from prize photographs collected by the members of the League, showing the impassable condition of earth roads, and also by cuts of some of the magnificent roads of Europe. As well show a picture of some of the hovels on the rocks on the north end of Manhattan Island or of a 46-story building, as representative buildings of New York City.

Second.—The article which has been quoted most frequently is one published by the United States Government, in which it is distinctly stated that the cost of wagon transportation in 1895 was $946 314 665.54

and that the possible annual saving by road improvement is $628 000 000. This article also distinctly asserts that the cost of hauling the farm crops to market is $662 000 000, or 26.6% of their value. Mr. Baker.

This report has been criticized in detail elsewhere, and hence needs no further examination here. With a certain class this conclusion had great influence, as it appeared to have the weight of the United States Government, and as it claimed to be the result obtained in answer to 10 000 letters sent to agricultural correspondents all over the country; but farmers of the rich grain-producing areas of the Mississippi Valley, the districts most in need of road improvement, protested loudly against any such estimate. They claimed that, through the exchange of work and by doing their hauling when other work was not pressing, the marketing of their crops cost them practically nothing.

Ten years after the above report was published, the United States Government published the results of another and more elaborate investigation, in which it is distinctly stated that the cost of hauling the 1905 crops to market was $84 684 000, or only one-eighth of the result in the former report! Other Government statistics show that the crops of 1905, the year of the last report, were practically twice as large as those of 1895, the year of the first report; and therefore the result by the later and more elaborate investigation is really only one-sixteenth of that of the first report! In other words, the result by the first report is virtually admitted to have been sixteen times too large! Is it any wonder that the farmer was unwilling to accept such a result?

As illustrating the fatality that seems to overtake hard-road enthusiasts when they attempt to compute the cost of bad roads, or rather as illustrating their proneness to look for excuses for making their results larger, the fact may be mentioned that in the latter investigation the author added $11 700 000 for the wheat that was hauled from the farm to the mill, which by his own figures is equivalent to saying that nearly one-third of all the wheat raised in this country is hauled in wagons from the farm to the mill. No statistics are at hand to check this item, but the probability is that the amount of wheat hauled in wagons to the mill is a very small proportion of the total; and hence this correction is much too large.

There are three other errors in the above investigation that make the conclusion too large: 1. The correspondent was asked: "What is the greatest distance of haul to shipping point by any considerable number of farmers?" The answer was assumed to be the radius of the contributing area, regardless of the fact that usually, because of topographic or other conditions, produce is hauled much farther from one direction than from another. This makes the distance of haul too great. 2. The average weight of load was assumed to be the mean

Mr. Baker. between the largest and the smallest, regardless of the fact that most of the farm products go to market when the earth roads are in nearly their best condition, and consequently the maximum load is very much more common than the minimum and not equally frequent as was assumed. This error makes the assumed average load too small, and therefore makes the computed cost of marketing farm products too great. 3. The price per day of team and driver was taken as the "usual cost of hiring a team and driver," regardless of the fact that hauling is a secondary employment to farmers and that the conditions of service, cost of feed and driver, loss of time by bad weather, etc., are very different for the farmer and the professional teamster. The farmer claims that the assumed price per day for team and driver is much too great.

The above total, $84 684 000, is probably at least twice too large. However, even this sum is only $15 each for the 5 740 000 farmers of the United States; and therefore the alleged cost of bad roads in marketing the crops is not likely to bankrupt the farmer. Of course, only part of this sum would be saved if the farmer had permanently hard roads upon which to haul; and consequently only a fractional part of this sum is available for hard-road construction, if only the economics of the problem is considered. Good roads are of an advantage to any rural community, but they must be defended chiefly for other than economic reasons.

Third.—The third article referred to is a speech by a United States Senator in the Senate in 1904, which was circulated as a public document. The Senator also traveled about over the country, apparently making the same speech. The speaker heard it twice in one day in the same room. The Senator said that an Illinois farmer owning 100 acres can get permanently hard roads for a tax of $20 per year for 5 years, provided the State or National Government will pay an equal amount; and then he will have three ways in which he can make 100% on his investment. The speaker will examine this statement briefly.

Under the most favorable assumptions, the above tax amounts to $1 280 per mile. What kind of a permanently hard road can be built anywhere for that sum? What kind of a road can be built for this sum in the Illinois corn belt, where the Senator was speaking and where gravel or broken stone must be hauled about 100 miles?

The Senator's first method of making 100% on the road tax is as follows:

"The farmer would get at least 52 days' labor, when not engaged in his crop, with his team at $2.50 per day which would amount to $130, of which half, or $65, would be clear profit."

In the first place, the tax is $100, but the Senator only claims $65 profit when he promised $100! In the second place, as the farmer and the Government together only pay $200, can 65% of it be safely spent for hauling the material?

Mr. Baker.

The Senator's second way of paying the road tax was by the reduced cost of hauling produce to market. In computing the cost on earth roads he virtually counted the cost of team and driver at $3.75 per day, even though he had just said that at $2.50 per day half was clear profit! He also counted the produce per acre more than four times that given by the U. S. Census. Granting his estimates and assumptions, he showed a profit under this head.

The Senator's third way of paying the road tax was by the increase in the value of the land. All hard-road advocates count both the reduced cost of transportation and the increase in the value of the land, regardless of the fact that the second is the result of the first, and also regardless of the further fact that hard roads add nothing to the productivity of the soil.

The Effect of the Good-Road Agitation.

Attention has been called to some of the absurdities of three of the most prominent pieces of good-road literature known to the speaker; and, if it were desirable, he could present others which are equally startling, although not of as good parentage. When some callow bicyclist writes on road reform for his local paper, or some automobile agent writes on roads for a trade journal, or a carriage dealer makes a speech at an annual convention, the statements of the three articles referred to are accepted because of their authorship, and the changes are rung upon their statements without any question as to their truthfulness. Accepting the above statistics and estimates as true, the conclusion is drawn that the farmer is a fool not to act thereon, and consequently such productions not infrequently bristle with opprobrious terms applied to the farmer.

Is it any wonder that the farmer has not been influenced, at least favorably, by such literature? What would be the effect if the dairymen, the corn growers and the cattle raisers should flood the cities with literature calling attention to the imperfections of street pavements, and claiming that the metropolitan residents were losing each year vast sums of money through lack of interest in the conditions of the pavements, and should cite noted boulevards and park drives as the kind of pavement the cities could and should have on all their streets?

Many of the advocates of hard-road construction have not had an adequate comprehension of the facts and figures they have presented, and have greatly underrated the understanding of the farmers they wished to convert. The men who make public opinion in any rural community know that, while good roads may have greatly enhanced the value of real estate in the bedroom of some great city, such conditions cannot be widespread. The representative farmer understands the difference between the conditions under which he

Mr. Baker. labors and those of the huckster near a large city. Even though hard roads may enable a farmer now and then to rush to town with a dozen of eggs, or a bushel of potatoes, or perhaps a load of hay, and obtain a fabulous price therefor, he knows that these conditions are exceptional; and also that, if any considerable number of hard roads are built, producers must accept the general level of prices. The Illinois farmer understands the difference between his State and Indiana in the matter of the proximity of road-building material; and he also understands the difference between the rich, sticky soil of his own State and that of Massachusetts, and believes that the experience in the Bay State is not a trustworthy guide for him. It may add spice to the article to embellish it with pictures showing wagons literally half buried in mud; but any man with sense knows that these conditions are not representative.

Within the past few years the agitation for hard roads has nearly ceased in the Prairie States, and, coincident with such subsidence, increased attention has been given to earth roads. When the hard-road agitation was actively going on, there was little or no demand in the farmers' institutes (meetings of the farmers to discuss agricultural topics) for articles or discussions on road topics; but recently there has been a large demand for information concerning the care of earth roads. This is very fortunate, for almost, if not absolutely, universally in this country the administration of road affairs is in the hands of small local official boards, which from the nature of our form of government are likely to change frequently. This condition imposes a well-nigh insurmountable limitation upon any comprehensive and continuous system of road improvement, unless the general public firmly believes in the value of the proposed system. Therefore it is highly important that correct information concerning road economics, road administration, and road construction should be widely disseminated. Unless a community is willing and able to maintain the earth roads in a reasonably good condition, it is useless to expect that it will be willing or able to support a high-class wagon road; and therefore the dissemination of correct information concerning the construction and care of earth roads is politically, economically, and physically the first step toward a better form of construction.

The problem of the earth road will now be considered briefly.

Drainage of Earth Roads.

Drainage is the most important matter to be considered in the construction of earth roads, since no road, whether of earth or stone, can long remain good without it. Water is the natural enemy of earth roads, for, mixed with dirt, it makes mud, and mud makes bad going. The rain or snow softens the earth; the horses' feet and the wagon

wheels mix and knead it; and soon the road becomes impassable mud, which the frost finally freezes, the second state of the road being worse than the first—for a time at least. Further, if the water is allowed to course down the middle of the road, it will wash away the earth, and leave gullies in the surface that must be laboriously filled up by traffic or by repairs. Prompt and thorough drainage is a vital essential in all road construction, and particularly so for earth roads. Mr. Baker.

For a road on loam or clay there are three systems of drainage, each of which must receive attention if the best results are to be obtained. These three systems are: (1) underdrainage, (2) side ditches, and (3) surface drainage.

Underdrainage.—Any soil in which the standing water in the ground comes at any season of the year within 4 or 5 ft. of the surface will be benefited by drainage; that is, if the soil does not have a natural underdrainage, it will be improved for road purposes by artificial subsurface drainage. It is the universal observation that roads in low places which are underdrained dry out sooner than undrained roads on higher land. Underdrained roads never get as bad as do those not so drained.

There are three distinct advantages of underdraining earth wagon roads.

The most important object of underdrainage is to lower the water level in the soil. The action of the sun and the breeze will finally dry the surface of the road; but if the foundation is soft and spongy, the wheels will wear ruts and the horses' feet will make depressions between the ruts. The first shower fills these depressions with water, and the road is soon a mass of mud. A good road cannot be maintained without a good foundation. An undrained soil is a poor foundation, while a dry subsoil can support almost any load.

A second object of underdrainage is to dry the ground quickly after a freeze. When the frost comes out of the ground in the spring, the thawing is quite as much from the bottom as from the top. If the land is underdrained, the water when released by thawing from below will be immediately carried away. This is particularly important in road drainage, since the foundation will then remain solid and the road itself will not be cut up. Underdrainage will usually prevent the "bottom dropping out" when the frost goes out of the ground.

A third, and sometimes a very important, object of subdrainage is to remove what may be called the underflow. In some places where the ground is comparatively dry when it freezes in the fall, it will be very wet in the spring when the frost comes out—surprisingly so, considering the dryness before freezing. The explanation is that, after the ground freezes, water rises slowly in the soil by the hydrostatic pressure of the water in higher places; and, if it is not

Mr. Baker. drawn off by underdrainage, it saturates the subsoil and rises as the frost goes out, so that the ground which was comparatively dry when it froze is practically saturated when it thaws.

The underdrainage of a road, not only removes the water, but prevents, or greatly reduces, the destructive effect of frost. The injurious effect of frost is caused entirely by the presence of water, and the more water there is in the roadbed the greater the injury to the road. The water expands on freezing, the surface is upheaved, and the soil is made porous; when thawing takes place, the ground is left honeycombed and spongy, ready to settle and sink, and under traffic the road "breaks up." If the road is kept dry, it will not break up. Underdrainage cannot prevent the surface of the road from becoming saturated with water during a rain; but it is the best means of removing the surplus water, thus drying the surface and preventing the subsequent heaving by frost.

Side Ditches.—The side ditches are to receive the water from the surface of the traveled way, and should carry it rapidly and entirely away from the roadside. They are useful, also, to intercept and carry off water that would otherwise flow from the side hills upon the road. Ordinarily, they need not be deep, but, if possible, should have a broad, flaring side toward the traveled way, to prevent accident if a vehicle should be crowded to the extreme side of the roadway. The outside bank should be flat enough to prevent caving. If the road is tiled, the side ditch need not be very large; but it should be of such a form as to permit its construction with the road machine or scraping grader, instead of by hand. On comparatively level ground, the proper form of side ditch is readily and cheaply made with the usual road machine or scraping grader. A deep, narrow ditch is expensive to maintain, since it is easily obstructed by the caving banks, by weeds, and by floating trash. If it is necessary to carry water along the side of the road through a rise in the ground, it is much better to lay a line of tile and nearly fill the ditch than to attempt to maintain a narrow, deep ditch.

The side ditch should have a uniform grade and a free outlet into some stream, so as to carry the water entirely away from the road. No good road can be obtained with side ditches that hold the water until it evaporates. Most ostensible road work is a positive damage, for this reason. Piling up the earth in the middle of the road is perhaps in itself well enough; but leaving undrained holes at the side probably more than counterbalances the benefits of the embankment. A road between long artificial ponds is always inferior, and is often impassable. It is cheaper and better to make a lower embankment, and to drain thoroughly the holes at the side of the road.

Surface Drainage.—The drainage of the surface of a road is very important, and is provided for by making the surface crowning and

keeping it smooth. It should be remembered that water upon the surface of the road cannot be carried away by the underdrains, since the water can reach them only after it has penetrated and softened the road surface. On account of the puddling of the surface when wet, by the action of the hoofs and wheels, and, under the most favorable conditions, little water will percolate through the surface and reach the tile, and, with clayey soil no water will be thus removed, the slope from the center to the side should be enough to carry the water freely and quickly to the side ditch; and, if the surface is kept free from ruts and holes, less crown will suffice than if no attention is given to keeping the surface smooth. If there is not enough crown, the water cannot easily reach the side ditches; and hence the road soon becomes water-soaked. Surface drainage is chiefly a matter of maintenance, and will be referred to again under that head. Mr. Baker.

Maintenance of Earth Roads.

The most important work in maintaining an earth road is to keep the surface smooth so that the rain-water will flow quickly into the side ditches. If the surface of the roadway is properly formed and kept smooth, the water will be shed into the side ditches and do comparatively little harm; but if it remains upon the surface, it will be absorbed and will convert the road into mud. If all ruts, depressions, and mud holes are not filled as soon as they appear, they will retain the water upon the surface, to be removed only by gradually soaking into the roadbed and by slowly evaporating; and each passing wheel or hoof will help to destroy the road.

There are several machines or devices which are very effective in filling ruts and depressions, and in keeping the surface smooth. Among these are the ordinary farm harrow, a square stick of timber shod on one edge with a strip of steel, a railroad rail, three or four 2-in. planks with their edges lapped and nailed, the **A**-drag, which consists of two planks set on edge vertically to form a letter **A**, and the split-log drag.

The road machine, or scraping grader, or road plane, a cutting blade suspended obliquely under a frame resting upon four wheels, is much used to smooth up the road in the spring; but it is not as good for this purpose as the more simple devices mentioned above. It is heavy, and cannot be used until the roads are too dry and hard for the most efficient work, and requires four, and usually six, horses and two men to operate it; while the other devices require only two horses and one man, and, if used at the proper time, are more effective.

The harrow is an efficient instrument in leveling the road just as the frost is going out, and also in smoothing the road in the summer when the surface has become rough.

Mr. Baker. All the other devices or machines act upon the same principle, that of filling up the ruts and depressions and working a little earth toward the center of the road to maintain the crown of the road by counteracting the wash of the rains. The timber or the railroad rail or the plank drag is drawn along the road with its length nearly perpendicular to the traveled way, but with the end toward the center of the road a little behind the other. By changing the angle of the cutting edge with the line of draft, according to the

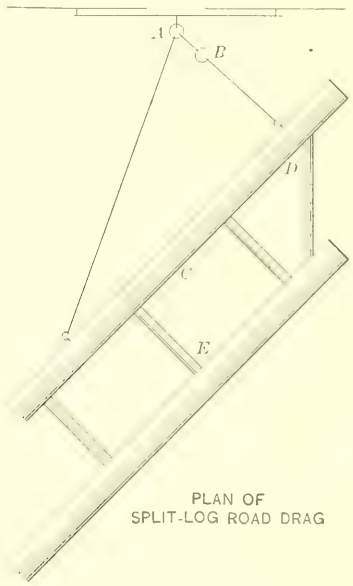

FIG. 3.

condition of the road, enough earth is pushed along in front to fill the ruts and depressions and also to work a little toward the center of the road to maintain the crown. The A-drag is drawn over the road with the pointed end forward. Some prefer this form of drag to the split-log drag; but there is not much difference either way, and, as the latter is more frequently used, it alone will be described in detail here.

Farmers in different parts of the country, for many years, have used some of these devices occasionally in smoothing the surface of the earth roads; but, of all these, none seems to have devised a better form of machine or been more persistent and intelligent in its use, or to have been more successful in interesting others in its use, than Mr. D. Ward King, of Maitland, Mo. Mr. King devised what he calls the split-log drag. A plan of the split-log drag is shown in Fig. 3, and Fig. 4 is a perspective view. The drag may be made

from a log 10 or 12 in. in diameter and from 7 to 9 ft. long. A light wood, like elm, is preferable to a heavy one, like oak. The cross-braces may be round or square sticks from 3 to 4 in. in diameter, the ends fitting into 2-in. auger holes. A board, not shown in the cut, is laid upon the cross-pieces for the driver to stand upon. The drag may also be made of two pieces of plank 10 or 12 in. wide and from 7 to 9 ft. long. The plank drag is shown in Fig. 5. It is wise to reinforce the wide planks with either a 1 by 6-in. or a 2 by 6-in.

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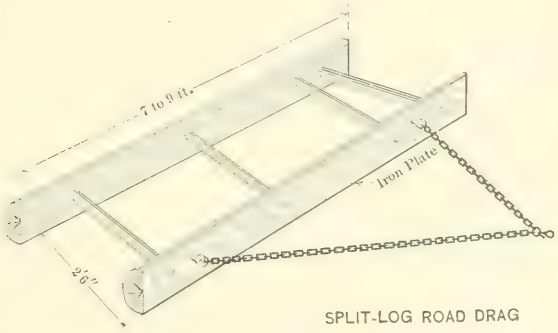

FIG. 4.

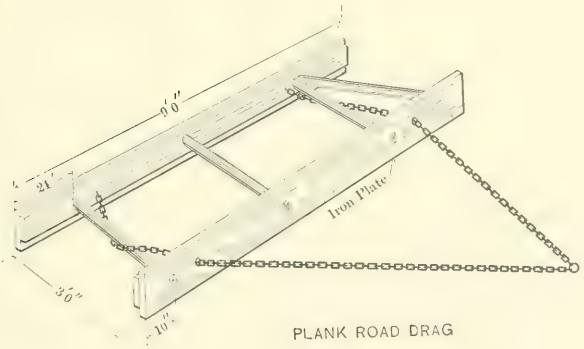

FIG. 5.

strip, as shown in Fig. 5. In Fig. 5 the chain is shown as being fastened on the rear side of the rear plank, which insures that the front plank will not be pulled off the cross-pieces—not a very important precaution.

The drag is drawn by two horses, and its length should be proportional to the weight of the horses. A drag 7 ft. long is about right for a team of 1 200-lb. horses, and one 9 ft. long for two 1 600-lb. horses. The driver rides upon the drag, and varies its effect by his position upon it. The drag does the best work when the soil is

Mr. Baker. moist, but not sticky. If the roadway is badly rutted and full of holes, it is well to drag it when the surface is slushy.

The use of the split-log drag improves the common earth road in three important ways: First, it smooths the road, which facilitates the drainage of the surface and also gives a better surface to drive upon. Second, the use of the drag moves earth toward the center of the road, thereby increasing or at least maintaining the crown of the road, which is necessary for good surface drainage. Third, if the drag is used when the surface is wet, the earth will be puddled, and then, when the sun shines, the earth will be baked—both of which will aid materially in making the road hard. Another valuable result of dragging the road when it is wet is the reduction of dust, for the particles of clay cohere more tenaciously and less dust is produced.

The cost of maintaining earth roads varies with the nature of the soil, with the weather, and with the degree of excellence attained; but, in a number of localities, roads on clay and on loam are greatly improved at an expense of from $3 to $5 per mile per annum, allowing from 30 to 40 cents per hour for driver and team. The work can be done to the best advantage by farmers who drag the road adjoining or near their own land, as it frequently happens that there is only a comparatively short time when the soil is in suitable condition for dragging. Fortunately, the best time to drag the roads is when it is too wet to work in the field.

Labor vs. Money Tax.

Road reformers usually grow eloquent in denouncing the labor tax system of caring for the public highways, and generally claim that the common earth roads are poor only because of the method of working out the tax. A little study of the facts and conditions will show that such claims are wide of the mark.

The labor-tax system is regularly used in all the States of the Union except five. In one State at least (Illinois) the road tax may be collected in money or labor as the township by election may decide, and a large majority of the towns vote in favor of the labor system. The labor-tax system was inherited from England, and is a survival of the feudal method of requiring all able-bodied men to render public service. England and France have a labor road-tax, but upon a much less extensive scale than the United States.

It is common to assume that the labor-tax system is all wrong, and that its evils would be escaped by paying road taxes in money. The labor tax has inherent disadvantages, but many of the defects charged to it belong rather to defective administration and to the system that leaves the control of the public highways to a small locally-governed community. Public work is seldom done as economically as private work.

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The objections to the labor-tax system are: 1. The labor is indifferent and inefficient. 2. It is impossible to get the work done at the most suitable time. 3. The system allows no selection of the laborer. All of these are important considerations.

The reply to the above objections is usually about as follows: 1. The farmer is willing to pay more in labor than in money, which compensates in part, at least, for the objections to the labor-tax system. This preference is not peculiar to the American farmer. In France, if the road tax is paid in money, a reduction of from 40 to 50% is made; but still 60% of the people prefer to pay in labor. Farmers not infrequently give more, both in labor and money, than is exacted as road taxes, because they are interested in better roads. 2. In many rural communities it is impossible to secure anyone to do road work at reasonable wages at the most suitable season. 3. If the tax were paid in money, there is no certainty that the labor would be any more efficient. Streets are maintained under the cash-tax system, but the labor is not ideally efficient. The authority that virtually wastes the labor tax will probably also waste the cash tax.

The labor tax is not necessarily the cause of inferior roads, nor the cash-tax system in itself the cause of improved roads. Townships under the labor-tax system often have better roads than adjoining townships under the cash-tax system. The one thing absolutely necessary for successful road management is effective supervision of the work. Without it, neither system will accomplish much, and, with it, either system will do reasonably well.

Many townships have changed from the labor-tax system to the cash-tax system with a marked improvement in the condition of the roads—due chiefly, if not wholly, to better administration. For in many of these cases the so-called cash-tax system is practically only a change in the method of administering the labor-tax system, since farmers desiring to do so are given an opportunity to work out their road taxes under the cash system. Under the labor-tax system those working upon the roads receive credit on their road taxes, while in the so-called cash system the laborer receives an order which is accepted as cash in paying taxes. In these cases the public sentiment that demanded road improvement secured the change from the labor tax to the cash tax; and, consciously or unconsciously, also secured a more efficient road administration.

Maintenance by Contract.

In view of the ordinarily inefficient system of caring for roads, it has frequently been proposed to maintain them by contract. As a rule, work done under the supervision of a contractor who has pecuniary interest in the result is more economical than that performed under the direction of a public official; but it is not wise to do work by contract un-

Mr. Baker. less the amount required can be approximately known beforehand, and also unless the character of the performance can be easily determined after completion. Neither of these important conditions would be present in a contract for the maintenance of a public highway. Owing to the indefiniteness as to the amount and character of the work to be done, it is not at all certain that the maintenance could be provided for by contract for a sum less than the public officials could do the work under the present system. The inspection would finally depend on the road official, and the letting of a contract would increase the difficulties and expense of supervision.

Under the present system, those who perform the road labor have an interest in the resulting condition of the roads, while the contractor would be interested only in doing the work for the least money; and therefore the roads would probably be worse under the contract system than under the present system.

It is claimed that the contractor could maintain a trained corps, and therefore do better work than can be obtained by the present system. This would possibly be true if the amount of work to be done were sufficiently great; but statistics show that the amount expended upon earth roads is only $40 or $50 per annum, and a large part of this is for bridges, which are built by contract, and a considerable part of the remainder is for tile and lumber for bridge floors, culverts, etc., while an additional sum is paid for laying tile and for dredged outlets, both of which usually represent contract work. Thus the amount remaining to be spent for the care of the traveled way and of the roadside is quite small; and therefore the ordinary expenditure for the care of earth roads is too small to justify maintaining a corps of expert road workmen. Further, leaving the road work to a comparatively few trained attendants would result in a great waste of time in traveling to and from the work. Again, the attendant would have so many miles of road under his care that he could visit any particular piece only at long intervals, and therefore could not do the work at the most favorable time, and could not become intimately acquainted with the road—conditions absolutely necessary for proper maintenance. These objections have less force as road expenditures increase, and as the money is concentrated on a comparatively few roads. Finally, a large proportion of the roads have an earth or gravel surface, and the labor required for their care is similar to that with which the farmer is familiar; and therefore he is not lacking in the skill required in maintaining them. The farmer who travels a particular road frequently and in all kinds of weather has a more intimate knowledge of it than the man who sees it only occasionally; and therefore, for this reason, the farmer is best able to care for the road. Besides, the farmer uses the road more than anybody else, and he alone pays for it.

The system of employing a man to give his entire time to the road is almost a necessity with first-class broken-stone roads, the maintenance of which requires intimate knowledge and constant attention, but the system is not applicable to earth roads. Mr. Baker.

Real Difficulties.

One of the most serious difficulties in the way of better earth roads is that the administration of road affairs is in the control of officers who are changed frequently and who devote only a small part of their time to the roads; and it is not clear that this difficulty can be overcome. It has been proposed to commit the general oversight of the roads to a county superintendent, much as the schools now are; but this plan is not very promising. The county superintendent of schools can hold examinations to determine the fitness of teachers; but a county superintendent of roads could not use such a method to control the efficiency of road workers. The county superintendent of schools can visit each teacher while at work, but a county superintendent of roads could hardly expect to do correspondingly for the road workers. A county superintendent of roads could render valuable public service in inspecting highway bridges and in preparing plans and letting contracts for new ones; and possibly might do something by holding public meetings for instruction in and discussion of road work, but it is not sure that the farmers could or would attend, and besides this function is already performed by the usual farmer's institute.

Another difficulty in the way of any great improvement of the common roads is that much of the richest agricultural land—that which needs better roads most and which is most able to pay for them—is held by non-resident owners; and the men who have opened farms and brought them to a high degree of cultivation, are leaving them and moving to the nearby small town or more remote city. The non-resident owner is not usually interested in the improvement of general conditions that do not bring him immediate financial returns; and the removal of the land owner lowers the general intellectual average of the rural community.

With the increasing introduction of agricultural machinery, the farms have been growing larger and larger and the number of residents in the country has been growing smaller and smaller, and consequently there are fewer people to be benefited by improved roads.

Signs of Promise for the Future.

Within the last few years the extravagant advocacy of hard roads has nearly ceased, and, almost coincident therewith, attention has been directed to the improvement of the earth road. This subject is con-

Mr. Baker. sidered at nearly every farmer's institute, and is treated in editorials and correspondence in the agricultural newspapers. Several of the States have State Highway Commissions which are devoting much time and money to experiments, to public addresses, and to the preparation and circulation of literature on the earth road.

Good roads add to the social, educational, and intellectual welfare of the rural community, facilitate freedom of intercourse between the dwellers on the farms and in the cities, and thereby contribute to the progress and stability of the country. Therefore, all should be willing to do all they can to help along the improvement in this line.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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IRRIGATION.

An Informal Discussion at the Annual Convention, June 25th, 1908.*

By Messrs. W. W. Follett, Gardner S. Williams, and F. C. Finkle.

W. W. Follett, M. Am. Soc. C. E.—*Imperial Valley.*—Picture to yourselves a desert—a desert so barren, so devoid of life that not a bush, a shrub, a blade of grass, nor even a cactus, can be seen, nor trace of animate nature in any form; a desolation broiling under the pitiless intensity of a semi-tropic sun. Such, seven years ago, was the greater portion of the Salton Desert, in Southern California, and the rest of it was but little better. Its eastern edge was some 40 miles west of the Colorado River, barren sand hills intervening. It was about 40 miles wide and 100 miles long, its lower end extending into Mexico. No fresh water was found within its bounds. Neither man nor beast could cross it without carrying drinking water. In its lowest portion, 287 ft. below sea level, was a deep deposit of salt, showing that it once formed an arm of the Gulf of California, and that, when shut off by the advancing delta of the Colorado River, the sea-water had evaporated. Mr. Follett.

For many years it had been a dream of several men to turn the waters of the Colorado into this desert for irrigation use, but only one man kept up the fight. Dogged, determined, C. R. Rockwood persisted in his efforts to impart to men with money his confidence in the future of the desert and in the feasibility of his plans.

A beautiful relief map of the Salton country appeared so often in various publications during the year prior to February, 1907, that it is doubtless as familiar to all as the face of an old friend. It will be remembered that Mr. Rockwood's scheme was to take water from the Colorado at Hanlon's, about 8 miles below Yuma, Arizona, by

* Continued from August, 1908, *Proceedings*.

Mr. Follett. a canal the bottom of which would be so far below the river's bed that the water would flow into it without the necessity for a dam. This canal would parallel the river for some 4 miles and then swing away from it and reach an old channel called the Alamo, 8 or 10 miles farther on. Down this channel, which leads to the Salton Sink, he proposed to carry water to a point, afterward named Sharp's Heading, some 35 or 40 miles down stream, or from 45 to 50 miles below Hanlon's. Here an earthen dam was to be built, the water was to be raised to the surface of the country, and a large system of main canals and laterals was to be constructed, by which the whole arable portion of the desert would be covered. Some 600 or 800 sq. miles of the lowest part of it is so salty as to be unfit for cultivation. A portion of this is now occupied by the Salton Sea.

For years Mr. Rockwood's efforts to interest capital were fruitless, but he kept on trying, discouraged but not dismayed by repeated failures. In his dreams, he saw a colony of prosperous homes and fertile farms irrigated by the Colorado's waters, rich with its fertile silt, and he named it the Imperial Valley. How many laughed at him and ridiculed his pretentious name, he alone knows. At last, after many disappointments, he formed, with some associates, the California Development Company, and conducted water to the desert in the spring of 1901, and his Imperial Valley sprang from the arid waste under its magic touch. So quickly did settlers come, and so rapidly was land brought into cultivation, that the canal people were at their wits' end to keep up the supply of water. The permanent Hanlon's gate had not been built, only a temporary heading existed, and the canal lacked several feet of being down to the proposed grade. In October, 1904, with more than 90 000 acres of land under cultivation and with 10 000 inhabitants in the valley, the heading of the canal began to silt up, and a water famine was imminent. An open cut was made from the river to the canal 4 miles below Hanlon's, and the world-famous Colorado crevasse resulted. It is not intended to go into the details of this crevasse, but, placed in Mr. Rockwood's position, nine out of ten engineers would have opened this cut just as he did—this is said advisedly, the speaker having an intimate knowledge of all the facts in the case.

It is now a matter of history that this crevasse was twice closed and that the second time it was held against the greatest flood which the Colorado has ever carried. These closures were accomplished only after a long struggle, extending over a year or more, and after several attempts had failed. The closures were made by H. T. Cory, M. Am. Soc. C. E. Behind him he had all the resources of a great transcontinental railroad and the help of a most efficient corps of assistants. He fought two veritable battles with the river. During their height, limited trains were side-tracked and held for hours in order that the movement of stone trains might be expedited; everything possible was done by the Southern Pacific Company which would hasten the work.

During all that year, the crevasse and its possible effects if it remained unclosed were exploited by able writers in newspapers, magazines, and proceedings of technical societies, nearly every writer stating or implying that its closure was impossible with the methods being used, and that, unless other supervision intervened, the Imperial Valley was doomed. The men on the ground said nothing, but sawed wood and quarried stone. A few of their friends had abiding faith in them. It would not do to admit even the chance of failure—too much was at stake. The potential value of the valley runs into the hundreds of millions. The crevasse was closed, and will probably remain closed. If, through some freak of the Colorado River, it should again be opened, so again will it be closed. Mr. Follett.

The Imperial Valley, with two broad and deep scars across its face, where the runaway waters flowed down Alamo and New River channels toward the sink and scoured out gorges from 40 to 60 ft. deep, and more than 1 000 ft. wide, is still flourishing. At no time during the existence of the crevasse was any of the cultivated valley deprived of water except that portion, some 15 000 acres, which was west of New River, although there was imminent danger all the time that certain waste-gates at the head of the Alamo would go out and lower the water level to such a point that no land could be irrigated. Heroic, well-directed, and persistent work in the face of great odds held these gates in place.

Only some 2 000 acres of land actually under cultivation were washed away, 400 acres of this being in Mexico.

These two deep channels, from 2 to 10 miles apart, running down through the lowest part of the valley, assure good drainage to the irrigated lands forevermore. Before the crevasse, the lands on the bottom of the valley were in danger of becoming water-logged. Now they are safe from that danger, and this safety alone repays the cost to the valley of the loss of area and all other losses.

Another advantage to the valley accruing from these cuts is the improved opportunity which they offer for the generation of electricity. At Holtville, on the Alamo, there was a power-plant prior to the crevasse, with a fall of some 40 ft.; this is now increased to more than 60 ft. At Calexico, where no chance for power formerly existed, there is now a sheer fall of 60 ft. into the bed of New River. It is evident that this is an opportunity for a large production of power. The tail-water, both on the Alamo and the New, can be picked up and used for irrigation before the Salton Sea is reached.

The valley is now booming. Settlers are going in at the rate of 1 000 per month, and the irrigated area is increasing so rapidly and in so many different places that it is difficult to give correct figures as to this year's area or products. The following statistics are probably under rather than over the facts.

There are between 18 000 and 20 000 people in the valley, and they

Mr. Follett. are watering this season 150 000 acres of land. They will export this year about $3 500 000 of products, besides their home consumption, and, while doing this, they will be incidentally extending the area of cultivated land, planting trees, building homes, and largely increasing their holdings of live stock and personal property. When it is remembered that at least 40% of this area was raw desert land a year ago, the full extent of the twelve months' work is seen.

To haul out this crop, the Southern Pacific Company has built upwards of 80 miles of track south of Imperial Junction. One small city, Imperial, and several thriving towns, Calexico, Brawley, El Centro, and Holtville, are found in the valley.

Look on this picture and then think of the conditions seven years ago, remembering that the valley went backward rather than forward during 1905 and 1906, and you will begin to realize what irrigation means to an arid country.

Nor is the story yet half told. It is a fetish in the orange country of California that oranges will grow only on a sloping porous ground with gravelly subsoil, and it was not supposed that they would mature in the Imperial Valley. Some venturesome man, however, planted a few trees at Imperial, and he has seen ripen on them the finest oranges one ever ate. So that the possibilities of this valley as an orange grove are so great as to become bewildering.

Owing to the advantage of the old Alamo channel, and of the opportunity offered by the rock point at Hanlon's for founding a headgate on bed-rock without a dam across the river, the construction of this enormous canal system has been comparatively cheap, even counting the cost of closing the crevasse. The only permanent structure on the system now, outside of the river levee, is the reinforced concrete heading at Hanlon's. It rests on solid rock, and its capacity is greater than will probably ever be needed. To complete the whole system, with permanent structures, and with canals extended to cover all the land available, will cost, together with the money now expended, less than $5 000 000. This will cover about 400 000 acres in California and 200 000 acres in Mexico—say that, in all, 500 000 acres are eventually irrigated. The cost is seen to be $10 per acre. The Reclamation Service projects run from $25 to $40 per acre. The low cost of the Imperial Valley scheme is due entirely to its favorable location.

Mr. Rockwood's Imperial Valley is no longer a dream. It is a pulsing reality, and is growing nearer to its name each year. In seven years more it will be a veritable empire.

Lower Rio Grande.—Now, let us go, with the crow, 1 500 miles south of east from Imperial to the lower reaches of the Rio Grande. Here the river has formed a delta extending for 100 miles back from the Gulf of Mexico—a delta of rich dark loam, of almost unknown depth, and of exceeding fertility. The rainfall is not sufficient to

mature a crop, but it is enough to cover the ground thickly with a jungle of arid-land vegetation, mesquit, catclaw and ebony trees, cacti of many varieties, and grama-, buffalo- and bunch-grass. Along the low bottom lands of the river are scattered Mexican ranches, where crops are raised from the moisture left in the ground after overflows. These ranches are very old, the Mexicans having lived there for many generations. Their livelihood is a precarious one, because no overflow means no crop, and the river does not top its banks every year. Mr. Follett.

The Rio Grande in this delta is not as large as the Colorado. Its maximum flood discharge at Brownsville is about 35 000 cu. ft. per sec., as compared with 110 000 cu. ft. per sec. for the Colorado at Yuma. Its minimum flow is seldom less than 1 800 cu. ft. per sec., except in rarely dry years. This is about one-half the minimum of the Colorado. Its mean low-water flow, on which irrigation projects may be based, is from 3 000 to 3 200 cu. ft. per sec.

Although this delta lies some 350 miles farther south than the Imperial Valley, its climate is better. The heat of summer is tempered by the Gulf breezes and probably by the dense covering of vegetation. Frosts occur in the winter, and sometimes cold "northers" make it disagreeable for a few days, but generally the winter climate is delightful.

It has been known for 10 or 12 years that sugar cane grows here luxuriantly and carries a large percentage of sugar. Rice also produces bountifully, if the ground is drained so as to be kept free from alkali. Alfalfa furnishes six or seven cuttings per year, and onions and other garden truck make enormous crops.

Within the past 2 years several different projects have been exploited for pumping water from the river and using it for irrigation. One company is taking out a gravity canal which is to be reinforced by pumping during low water. The Rio Grande forms the boundary between the United States and Mexico, and all the projects here mentioned lie north of the river, or in the United States. There is much more land available than water. Mexico is entitled to half the latter; so that all the land cannot be watered, and the best on each side should be chosen. These delta lands should be preferred to bench lands higher up the river, and, in the United States, perennial irrigation for, say, 150 000 acres and flood irrigation for as much more is the extreme which can be expected.

In addition to raising the crops previously mentioned, it may prove to be a good grape and nut country, but this is not yet demonstrated. The frosts will probably preclude the growth of citrus fruits.

With the exception of one company whose works, built some 5 years ago, were mismanaged and proved to be a failure temporarily, none of the sixteen companies, now putting in plants, is more than 2 years old. Indeed, none of the plants is yet completed, yet this year they are water-

Mr. Follett. ing 15 000 acres, and will water three times as much next year, and the increase will continue for several years. The water is lifted by centrifugal pumps, the lift when the river is low averaging about 16 ft. Pumps having a discharge up to 36 in. are in use. The latter are rated to throw 45 000 gal. per min., or 100 cu. ft. per sec. Wood or oil is used for fuel, and the pumping is done cheaply.

The amount of land now watered is comparatively small, but the increase will be rapid, and the cultivation is intensive. There is direct rail connection with the northern and eastern markets, and hardy vegetables can be grown all the year around.

Last winter this country had a veritable invasion of home seekers. Many bought land, and the boom is on, but it is too soon to show such specific results as does the Imperial Valley. Enough has been done, however, to prove that it will be a great sugar-producing country, and large quantities of other products will be raised. The "back country" for 100 miles north of the river is the great Texas cattle-breeding ground, and there is no reason why this stock should not be fattened on alfalfa grown along the river instead of being shipped north to eat Kansas and Nebraska corn. The potentiality of this section is enormous. Although it has lain dormant for generations, the awakening has come, and progress will now be rapid. Nothing on earth can compare with the wonderful metamorphosis which irrigation development produces.

Both the Imperial Valley and the Brownsville country are being developed by private capital. The Federal Government has not contributed to the work in any way.

The Rio Grande Project.—The Rio Grande is a peculiar stream. Its source in Colorado is high up on the Continental Divide. It first runs east and then south through the San Luis Valley, with its 400 000 acres of irrigated land, a plateau from 7 500 to 8 000 ft. above sea level and surrounded by lofty snow-capped mountain ranges several thousand feet higher. The river enters New Mexico, still among the snow mountains, and continues in them until it passes Santa Fe. In the extreme northern end of the San Luis Valley there is an area of some 2 600 sq. miles—about half mountain and half plain—which contributes no run-off to the main river. The remainder of the area above described, as far south as Santa Fe, N. M., has a large snow fall and furnishes much water.

Below Santa Fe, no living streams enter the river for a distance of more than 500 miles. There are several torrential channels which occasionally discharge for a short time large volumes of water heavily loaded with silt. El Paso lies near the lower end of the middle third of this strip. From this point to its mouth the river forms the boundary between the United States and Mexico.

At the lower end of this 500-mile section, there enters from Mexico

the Conchos River. This is a torrential stream, discharging occasionally enormous quantities of water. It also has a small perennial flow. Below the Conchos, the river enters a cañon many hundred miles long. In it are numerous springs which, together with the Conchos and two or three other tributaries, make up quite a stream. Its maximum flood waves are at Eagle Pass, where 240 000 cu. ft. per sec. have run for 24 hours. Below, the waves flatten out, and the water ponds in old channels and on low ground, so that, at Brownsville, as before stated, the maximum flood is 35 000 cu. ft. per sec. The total annual discharge here, however, is greater than at Eagle Pass. The total length of channel, from its source to the Gulf of Mexico, measured along its sinuosities, is about 2 000 miles. Mr. Follett.

In New Mexico, along the Rio Grande and its tributaries, exist the oldest irrigation works on the American Continent which are still in use. When the Spaniards entered New Mexico, more than 350 years ago, they found the Pueblo Indians, living in their many-storied towns and cultivating the land of the valleys, bringing water to it by acequias or irrigating ditches. Many of these are in use to this day. How long these Indians had been there is unknown, but they were then old inhabitants. There are 17 or 18 of these pueblos. The one lowest down the river is at Isleta, about 15 miles below Albuquerque. Below here, as far down as Socorro, ditches were taken out by the Spaniards at an early date, but the Jornada del Muerto and the fertile Mesilla Valley were the stamping ground of the Apache Indians until comparatively recent years. The first ditch in the Mesilla Valley was built in 1844. In the El Paso Valley, below the Mesilla, the Indians were driven out, and irrigation began in the latter part of the sixteenth century.

These settlements were very prosperous until the early Eighties of the nineteenth century. Then the large development of irrigation in the San Luis Valley began to affect the Rio Grande above the mouth of the Conchos, and since that time it has frequently gone dry and remained so for months. No water reached the Conchos for 20 months prior to May, 1900. At El Paso, the river was dry, or practically so, for 227 days out of 365 in 1902. It was practically dry (carrying less than 15 cu. ft. per sec.) from January 9th to August 8th, 1904—213 days of continuous drouth. Of this time the record shows "no flow" (which means less than 2½ cu. ft. per sec. or none) from March 1st to August 8th—161 days. In September, 1896, the speaker saw weeds 2 ft. high growing clear across the bed of the river above the mouth of the Puerco, more than 200 miles above El Paso and about 35 miles below Albuquerque. The river was then dry as far north as Albuquerque. Its flood discharge at El Paso almost reaches 25 000 cu. ft. per sec., and at San Marcial it exceeds 30 000 cu. ft. per sec. The total discharge at El Paso in 1902 was 50 800 acre-ft. In 1903 it was 1 033 000 acre-ft., and in 1905 it was more than 2 000 000 acre-ft.

Mr. Follett. These figures show that it is here a torrential stream, and that, to obtain the best economic results from the use of its waters for irrrigation, they must be stored. Fortunately, there exists an available reservoir site in which can be stored 2 000 000 acre-ft. of water—possibly 2 500 000 acre-ft. This is located about 125 miles above El Paso, 10 miles west of Engle, N. M., and some 60 miles below San Marcial. To build a dam and the necessary distribution system will cost upward of $8 000 000. The land on which this water will be used lies in the Territory of New Mexico, the State of Texas, and the Republic of Mexico. The first cost is too great for individual capital to attempt. The diversity of interests is too complicated for individual or corporate powers to handle. But the Reclamation Service, under the able direction of F. H. Newell, M. Am. Soc. C. E., can handle it, and has taken hold of the enterprise. Through the exertions of one of its best-natured engineers, B. M. Hall, M. Am. Soc. C. E., the antagonisms of many years between the different interests were allayed, and all the communities interested are now working in harmony. Two Water Users' Associations, one in New Mexico and one in Texas, have been legally formed, and the lands have been signed up under the Reclamation Act. All that the land owners can do has been done.

Across the river from El Paso lies the Mexican settlement of Juarez. Many years ago it was noted for its grapes and other fruits, and its large crops of wheat. The records in the old Juarez church show that the Acequia Madre was in existence and doing business more than 300 years ago. When the dry river came, the fruit trees and the vines died for lack of water, and thousands of acres of cultivated land were abandoned. The population dwindled to a fourth of its former number. The Mexicans felt themselves aggrieved in that their water, theirs for centuries, the breath of their life, the one thing which gave value to their lands, had been taken from them. The United States Government recognized their moral, if not legal, claim, and by solemn treaty has agreed to deliver to them, of the water to be stored in the Engle Reservoir, 60 000 acre-ft. each year, and has appropriated from general funds $1 000 000 toward the $8 000 000 required for completing this vast Rio Grande project. The remainder will come from the Reclamation fund when it is needed. Thus, this great enterprise is under way. Already, a diversion dam, costing, with its appurtenances, $200 000, has been built at the head of the Mesilla Valley, and is now in use. At the Engle site a rubble concrete dam, 270 ft. or more from bed-rock to coping, and 1 200 ft. long on top, will, it is hoped, soon begin to grow.

More than 200 000 acres of fertile alluvial land will be watered, and its products, being in the center of the arid country and near large mining camps, will find ready sale. To-day there is an attempt made to irrigate some 50 000 acres of land scattered in the long, narrow valleys stretching from Engle to old Fort Hancock, 175 miles down stream,

but the result of planting is uncertain, especially below El Paso, and the sparse population has a precarious livelihood; but, a few years from now—it may be 5 and it may possibly be 10—these valleys will teem with happy homes. The farmer will put seed into the ground with a surety that it will fructify and return a bountiful harvest, because he knows that the water to produce his crop is already stored in the reservoir up the river. His dependence is not in seasonable showers or in uncertain river flow, but in that stored water, and it is safe. Mr. Follett.

This reservoir is made so large in proportion to the area to be irrigated for two reasons: First, a succession of dry seasons may necessitate carrying water over for several years. Thus, it would have been necessary to have held some of the 1896 flow until 1902, in order to have exploited fully the capacity of the stream. Second, space must be given for the deposit of silt. While it is believed that plans are developing which will take care of all the sand moving on the bottom of the stream and of a large part of the silt in suspension, carrying it out of the reservoir, some silt will remain, and extra space must be provided for it. It is believed that this reservoir can be constructed so as to supply 200 000 acres indefinitely, and that silt as well as water can be furnished to the land, thus keeping up its fertility.

Do you all realize what these figures mean? Two hundred thousand acres does not seem so much when you write it down, but, when you from the East start home, remember that if this land were in a strip one mile wide with the railroad through the center, and all were irrigated, you would ride for more than 300 miles through a veritable garden. If you were to go by water, and were traveling on a steamer drawing 15 ft., these 2 000 000 acre-ft. would furnish water for a channel all the way to New York, with 5 ft. under the keel and a waterway nearly 450 ft. wide.

This will be by far the largest artificial lake in the world. At present an Indian reservoir, which is filled only once in 20 years or so, is the largest, with a capacity of 950 000 acre-ft. Then comes the Assuan Reservoir, on the Nile, which, as now constructed, holds 900 000 acre-ft. When completed, the Roosevelt Dam, now being built by the United States Reclamation Service, on Salt River, in Arizona, will impound 1 300 000 acre-ft. of water, and thus will hold the record of being the largest artificial reservoir in the world, until the completion of the Engle Dam.

A Few Indian Irrigation Works.—For the sake of comparison, let it be recalled that, in far-away India, the English engineers have built, and are now quietly building, irrigation systems having a magnitude beside which the largest in the United States look almost insignificant, as far as acreage covered, and size and length of canals, are concerned. The speaker has collected the following information from the 1905 edition of Buckley's "The Irrigation Works of India."

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In 1903, in British India, more than 44 000 000 acres of land were irrigated, of which about 20 000 000 were watered from canals built by the engineers, and from the funds of the British Government. Of this latter quantity, more than 3 600 000 acres were watered by three canals taken from branches of the Indus, in the Punjab. The Bari Doab, taken from the Ravi, carries 4 500 cu. ft. per sec., and waters 850 000 acres. The Sirhind, taken from the Sutlej, has a capacity of 8 200 cu. ft. per sec., and, in 1903, watered 1 170 000 acres. The Chenab heading, in the river of the same name, has a capacity of 10 800 cu. ft. per sec., and, in 1903, served 1 600 000 acres. Its mean flow during the rainy season was more than 8 000 cu. ft. per sec., and, during the dry season, 6 000 cu. ft. per sec. It is said to have watered in one year 2 000 000 acres, or as much land as is now irrigated in the whole State of Colorado. It covers more than 2 600 000 acres, and cost to build about $10 000 000, or less than $4 per acre. This low cost is doubtless partly due to natural advantages of the country and partly to cheap labor. It paid 21% on its cost in 1903, clear of the cost of collecting rentals and of maintenance.

On the Ganges River, the Upper Ganges Canal carries 8 000 cu. ft. per sec. It serves not only to irrigate 1 300 000 acres of land, but also acts as a feeder to the Lower Ganges Canal. The latter carries 5 000 cu. ft. per sec., and, with the help of the upper canal, waters more than 1 200 000 acres. Water, taken from the river by the upper canal, is carried 180 miles before it is turned into the lower canal, and then travels 240 miles farther before it is all used. The irrigator on the delta at the junction of the Ganges and Jumna Rivers turns on his land water which has been transported 420 miles in an artificial earthen channel.

Up to 1903, the British Government had spent about $150 000 000 on canal systems, or an average of $7.50 per acre for the land actually watered. In that year, it derived an average net revenue of 6.3% on the cost. Many systems were operated at a loss, and will probably be so for many years. They are intended to ameliorate famine conditions. Two large projects were then approved and funds provided for them. These would cover upward of 5 000 000 acres, and are now doubtless completed and in use.

Stupendous as these projects are, they do not mean so much in proportion to size as do those in the United States in adding to human advancement. It is stated that the Chenab has made available homes for 1 000 000 people—or two inhabitants for every three acres irrigated. Some of the other systems have a denser population, and their construction has served principally to lessen the death rate from famine. They have ameliorated the condition of the natives, but have not brought in so high a civilization as do smaller enterprises in the United States, where homes are made, universities founded, churches built, and all the

complexities of a higher civilization set under way. In Colorado may be seen what 40 years have done in reclaiming the desert and, although the total acreage is less than one-third of that watered by the Chenab alone, a vast addition to the happiness of mankind may be observed. Mr. Follett.

The Law of Prior Appropriation.—In the humid States the old common law of riparian rights obtains. Every man owning a frontage upon a living stream has the right in perpetuity of undiminished flow in it. He may use the water in any way he wishes, as for power purposes, provided he does not injure his neighbors above by flowing their lands, and provided he returns to the stream below the water practically undiminished in volume.

This law will not serve the arid country, where water, taken from a stream for irrigation, returns no more to it except, in some cases, in greatly diminished volume, as seepage. Should the tenets of riparian law be enforced, irrigation would stop. Therefore, a new principle was evolved from the necessity of the situation—the law of prior appropriation. The water of all streams is declared the property of the public or of the State, subject to appropriation for beneficial use, and he who first appropriates all or a portion of the water of any stream and perfects his claim by putting it to a beneficial use, obtains a legal property right to that water. It becomes his as much as is the land on which he uses it for irrigation, if this be the purpose of his appropriation, and such it usually is.

You may own the land on both banks of a stream for miles, and yet, should you allow its waters to go to waste, a neighbor below you can enter on your land, locate and build a dam and a headgate in the stream, condemn a right-of-way for his ditch across your land, build it, and divert that water to his lands below, and you lose all rights to it.

Where several appropriators take water from the same stream, the one who first took it must have what water he needs, up to the amount of his appropriation, before the second is allowed any, and so on up the list. These rights are determined by different methods in the different States, but they are all called "adjudications." After a stream is adjudicated, the water is distributed among the several ditches by a State official.

Thus far, this adjudication has been confined to that portion of interstate streams which lies in each State, all rights of the State lower down being ignored by the one above it, but there is no logical reason why an imaginary line across a stream should give the land owner above that line a right to take the property of the man below it, whether it be a county, State, or international boundary. For, in an irrigated country, the land has no value without the water, and the property right vests in the latter. If a man's water is taken from him, he is robbed, whether or not an imaginary line crosses the stream of supply between him and the robber.

Mr. Follett. There are many ramifications of this law, and it is materially modified in different States, but the basic principle applies to the water laws of all the arid and semi-arid States except to those of California. There, the riparian theory ostensibly holds, but it has been modified to such an extent by statutes and Court decisions that it is unrecognizable.

Those interested in this phase of irrigation will find the subject fully discussed in "Irrigation Institutions," by Elwood Mead, M. Am. Soc. C. E.*

It has never seemed to the speaker that the storage of water for the development of electrical power and for irrigation go together. The supply of stored water for electrical development must be one which is continuous throughout the whole low-water season of the stream which it supplements. Water stored for irrigation will be used within a comparatively short time. It may be stored and held for several months, or even, in the case of large reservoirs, for years, and then all used in two or three months. This will not serve for electrical power development.

Now, as to seepage: The seepage return in Colorado has been carefully studied by Professor Carpenter and others, and it exists, as has been stated. On the Rio Grande, however, there is no seepage return from irrigation below the San Luis Valley. In that valley there is a small return in some places and a large loss from seepage in others—as on the main Rio Grande between Del Norte and Monte Vista. In the foregoing paper the speaker mentioned having seen weeds growing 2 ft. high across the bed of the Rio Grande below Albuquerque, in September, 1896. During that season, and for 200 years prior to that time, there had been, in the 40 miles of the valley of the river above the point where these weeds were growing, some 20 000 acres of land under cultivation. It had been irrigated every year, and still there was not one drop of seepage return from that irrigation water going into the channel of the stream.

The whole Rio Grande Valley is an alluvial deposit. The river in most places lies on the top of the valley, and the water, once taken from the stream, never comes back. A few days after those weeds were seen, the speaker visited a point, a hundred miles or more down the river, where there was a reef of rock which crossed the river bed and probably forced to the surface all the underflow which might be going down the stream. When visited, there were about 4 cu. ft. per sec. running, the river being dry above that point.

In 1903 Professor Slichter, who has measured the velocity of underground waters in various places for the Reclamation Service, came to El Paso and attempted to measure the velocity of the underflow in the Rio Grande where it runs through "the Pass" just above

*The above paper was read to the meeting of June 24th, 1908; the discussion which follows was contributed by Mr. Follett at the meeting of June 25th, 1908.

that town. There the hills shut the river into a space about 350 ft. wide, with bed-rock 70 to 90 ft. below the bed of the stream. He could not detect any velocity. Mr. Follett.

The question naturally arises, What becomes of the waste water from irrigation and also from overflow? The only answer which the speaker can give is, that it evidently sinks into the ground and is held there until it is evaporated or transpired through growing vegetation. The material of the valley is fine sand and alluvium interspersed with thin layers of clay. Water does not percolate through it at any appreciable rate of speed. The river is a torrential stream, becoming dry at times, and irrigation goes on for only six or seven months of the year. The rainfall is 8 to 10 in. per annum, and the evaporation from a pan in water is about 6 ft. per year plus the rainfall; so that, between floods and between irrigation seasons, the subsurface water is dispersed in the air to such an extent that room is made for the next year's surplus.

In this discussion it has been stated, as a general economic principle, that water should be stored for the development of electrical power and then used for irrigation, and the inference was that the water was stored but once. When the speaker challenged this statement, it was then said that two reservoirs must be used, one above the power station and another below, and that the lower reservoir need be only one-third as large as the upper, provided the system could be started off with the lower reservoir full.

After considerable study of the problem, the speaker is unable to accept this statement. He does not believe that the scheme will work in practice, especially on streams whose perennial flow is overappropriated for direct irrigation and whose flood supply, in seasons of small spring flow, is overappropriated for storage of water to be used later for irrigation. The first condition obtains on every stream on the eastern slope of the Rockies, south of Wyoming, and the latter exists on many. Perhaps the statement of a concrete example will make the speaker's position plainer.

The Cache la Poudre, which supplies the Fort Collins and Greeley Country, had a deficient flood flow this spring, and, as a result, the numerous reservoirs on the plains, holding storage rights to flood water, were only one-half to one-third filled. The full capacity of these reservoirs is always needed in August to mature the potato crop and to irrigate alfalfa.

Now, with these conditions existing, assume that a large reservoir had been built up the stream in the mountains for power storage and had been filled with water during the spring of 1908. Under the law of prior appropriation, this water would be the property of the reservoir owners on the plains, as their reservoirs were unfilled. The power company would need the water to supplement the low-water flow of the stream throughout the whole period of small flow.

Mr. Follett. This period runs from the latter part of June or early in July until February or March, and the draft for power on the mountain reservoir should be so proportioned, month by month, as to make the water last for the seven or eight months during which it may be needed.

Owing to the shortage of water, all the plains reservoirs become empty in August, and crops are suffering and dying for want of water. The power reservoir is filled with water to 70% or more of its capacity. This water, had it not been stored there, would have been held in the plains reservoirs and would have been used toward saving the crops. Under these conditions, it is almost certain that the Water Commissioner for that district would compel the power people to turn that water loose, and let it all come down the stream in two or three weeks' time. By September 1st the reservoir would be dry, and the power plant would be crippled for the next six months. No small secondary reservoir would be of any material service.

It may be claimed that this is an exaggerated case. Perhaps it is, but it illustrates clearly the speaker's position, and is one which may be, and will be, approximated on any stream of the Gulf drainage from the Continental Divide.

Where power plants are developed with the storage of flood waters not already appropriated, their operation will result in increasing the perennial flow of the stream below their wheels and so will benefit irrigation. If it is desired to store the flow from the plant after the cropping season is over, for use the following year, this can be done if reservoir sites can be found; but the speaker would consider this an entirely distinct development independent of the power plant, and one which would be properly undertaken by those to be benefited by the storage and not by the power company as such.

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E.—There seem to be rather conflicting ideas on the matter of return water; but, to some extent, they can be explained quite satisfactorily in the light of what is known of the absorption of water by various kinds of plants, and the variable amount of evaporation from different soils. It may be suggested that in the Rio Grande Valley the quantity of water spread upon the land has not been sufficiently great to leave any excess over that needed by the vegetation and that evaporated directly from the soil; whereas, in other localities there may be so much difference in the situation that the demands of these two agencies for water have left some over to fill up the subterranean basins, and that possibility may account for the difference in the return water from the several drainage areas. It may even happen, as time goes on, if the soil be sufficiently porous, that a part of the water used for irrigation purposes in the Rio Grande Valley will return to the streams.

Mr. Finkle. F. C. FINKLE, ESQ.—Mr. Follett has stated that the development of electric power may be greatly restricted, or entirely prohibited, in many

sections of the arid region, because of the conflict with irrigation. Mr. Finkle.
He bases his conclusion on the conflicting nature of the uses of water from streams for irrigation and power. It is true that water for irrigation must be available during the months of seeding, sprouting, and growing, and that it must be conserved during the remainder of the year. Usually, five or six months of each year cover the irrigation season, and these are during the spring and summer. Throughout the arid West the only exception to this is in the sections of Southern California where citrus fruits or winter fruits and vegetables are grown. For these crops it sometimes becomes necessary to irrigate every month in the year, during seasons of deficient rainfall, or "dry years."

On the other hand, water for power purposes must be continuous throughout the year; must, in fact, be more plentiful in autumn and winter because days are shorter and more power is consumed for illumination.

The mountain streams suitable for developing power have their greatest flow in summer, usually become very low in the fall, and attain their minimum flow in winter. This arrangement is favorable to irrigation, except when the snows melt so rapidly as to cause great floods in the early spring and a scarcity during the summer growing season.

If such streams are equalized, so as to make their use profitable for the development of power, the effect will be two-fold, as far as irrigation is concerned. The large floods may be stored in reservoirs and released when the stream naturally discharges less water than is needed to operate the power-plant. This does not interfere with irrigation. Or, if the power development is planned on a larger scale, it is necessary to store some of the normal spring and summer flow to regulate the streams to a higher flow during the minimum months. This will interfere with irrigation unless measures are taken to return the water thus impounded—when it is necessary for irrigation—at some subsequent time, and it will then have an equal value for that purpose.

Such a result can be accomplished by the use of secondary storage below the tail-race of the power-plant. This will amount to double storage for a part of the water used by the power-plant.

To illustrate: For a stream having a discharge of 200 or 300 cu. ft. per sec. during May, June, July, and August, and a diminished flow, as low as 20 cu. ft. per sec., during December, January, February, and March, it is necessary to store water at the headwaters, so as to regulate it to give a flow of not less than 100 cu. ft. per sec. during the whole year. If the storage is available and accomplishes this, the surplus over 100 cu. ft. per sec., which is demanded continuously for the power-plant, must be stored during

Mr. Finkle. May, June, July, and August. Now, if all the water is required for irrigation during these months, it is clear that the storage of any part of it will interfere with its use for that purpose. It will be necessary, therefore, to construct another reservoir, at some point below the tail-race of the power-plant and above the intake of the irrigation canals, by means of which the water used in the power-plant can be impounded during the non-irrigation months, and released during the following irrigation season. This is called a secondary reservoir, and the purpose it accomplishes is called secondary storage. After it is once filled, its operation effectually prevents any conflict between the power and irrigation interests on the stream. This operation merely consists in releasing contemporaneously from the secondary reservoir the same quantity of water as that taken to fill the reservoir above the intake of the power-plant.

During the months of low flow in the stream, the secondary reservoir is again filled from water passing through the power-plant, which at that time is not demanded for any other purpose than that of generating power. Thus the process becomes an endless chain, by which the natural flow of the stream is maintained for the benefit of irrigation, and at the same time it is regulated for use as required by the power-plant.

No difficulty and interference with irrigation need be anticipated when first filling the secondary reservoir, as this may be done from water flowing during the non-irrigation months, while the power-plant is being constructed.

There are examples of power-plants, where this method is used to harmonize the interests of users for power and irrigation on the same stream.

To illustrate this the speaker can refer to those plants of the Edison Electric Company, of Los Angeles, on Mill Creek, in San Bernardino County, California, known as "Mill Creek No. 1" and "Mill Creek No. 3," which he designed and constructed for that company many years ago.

The entire flow of Mill Creek is used for the irrigation of valuable citrus orchards near Redlands, Cal. These water rights are among the oldest in the State, having been appropriated by the Indians for the irrigation of their gardens and corn fields sometime before 1826. They passed later to the Mexicans and from them to the Americans, being at the present time valued as high as $100 000 for each cubic foot per second of perpetual flow of water.

The use of the water for power would not be profitable unless regulation of some kind by storage could be accomplished, because the flow of the stream is extremely variable and not adapted to the load curve obtainable for a power-plant. Yet no storage could be permitted which would alter the natural discharge of the stream without infringing on these very valuable irrigation water rights.

Mr. Finkle.

The problem was solved by constructing a primary reservoir above the head of the force main for the upper power-plant. From this reservoir the water is used through the upper plant, under a head of 1 923 ft., and then through the lower one, under a head of 530 ft. Below the tail-race of the lower plant and above the point of diversion of the "Mill Creek Zanja," which is the main conduit of the irrigation system, is located the secondary reservoir. By means of this reservoir the stream is regulated back to its natural flow.

This system has been in operation for many years, and has worked successfully. No injury has resulted to the irrigators, and the power-plants have not been hampered by conflicts over vested irrigation rights, nor deprived of the water required for their business.

Acting on the same principle, the Eastern Colorado Power Company is now constructing a power-plant on Middle Boulder Creek, about 30 miles from Denver. In connection with this development, storage reservoirs in the mountains are being constructed to impound water in summer, which will be turned through the conduit to the power-plant for use in winter, when the natural stream flow is wholly insufficient.

In order that the water used for power in winter may not be taken away from land requiring its use for irrigation, the company is preparing to build a secondary reservoir below the power-house. By means of this reservoir, what has already been explained will be accomplished, namely, re-storage of the water used for power in winter and its use for irrigation during the following summer.

A report has been made for a similar power development for the Eastern Colorado Power Company. This is on the Big Thompson River, and also on the eastern slope of the Rocky Mountains. Several excellent reservoir sites in the mountains are available, and locations for two power-plants below these have been recommended. For secondary storage, Boyd Lake Reservoir, referred to by Mr. Anderson, has been selected. This reservoir has been practically completed by Mr. B. D. Sanborn, a Greeley capitalist, who has given an option on a portion of its capacity to be used by the Eastern Colorado Power Company as secondary storage in connection with the Big Thompson power projects.

The principles thus applied to practical use are necessary in carrying out power projects on the eastern slope, in Colorado. The streams flowing from the eastern slope of the Rocky Mountains are either completely used for irrigation, or the demand of the future is so great as to make their conservation for this purpose a necessity. It is apparent, therefore, that power-plants in this region must be planned with double storage.

Time has only permitted a discussion of the general principles involved, but it may well be added, that the secondary storage need

Mr. Finkle. not be as large as the primary. This is a detail which must be carefully worked out from the data presented by each individual case.

Mr. Newell has referred to another very important matter for the irrigation engineer, that of concrete work failing under certain conditions.

An instance of this was observed about 10 years ago, at Santa Barbara, Cal., where a concrete pipe became completely disintegrated. The pipe was made originally from the best hydraulic Portland cement, all carefully tested, and the aggregates were of the best quality of sand and gravel. The workmanship was the same as on similar pipes which have been used for more than thirty years and are apparently unaffected by time. Investigation revealed only one possible cause for failure: The soil in which the pipe was laid contained iron pyrites, which by decomposition produced sulphuretted hydrogen, and the presumption is that this affected the cement.

A similar failure of concrete pipe occurred at Cucamonga, Cal., a few years ago. The pipe had been properly constructed, and was laid in a swamp. Examinations of the soil and water revealed only one theory by which the softening of the concrete could be explained. The conclusion that decomposition and fermentation of the vegetable matter in the swamp produced carbon dioxide, which caused the trouble, seemed inevitable. It is known that carbonic acid gas has a strong affinity for lime, and it is supposed that its combination with the lime in the cement produced carbonate of lime. The cement in the pipe had become soft, and resembled talc, or a mixture of clay and limestone dust. To all appearances, it had returned to the state in which it was found before it was burned into cement.

Cement is supposed to be one of the most indestructible materials, and yet it cannot be used under all conditions.

It is well known that steel, iron and wood must be used with great discretion in order to build works of a permanent character. Referring to wood, and particularly as a material for water pipes, it is interesting to recall the paper* by Arthur L. Adams, M. Am. Soc. C. E., regarding the water-works at Astoria, Ore.

The case cited by Mr. Adams is one of several where wooden-stave pipes have been unsuccessful, while there are many other instances in which they have been entirely successful. The failure at Astoria is undoubtedly due to the large quantity of vegetable matter in the soil, which makes it a "muck" soil. This conclusion is borne out by experiments made elsewhere to learn the effect on wood in soils where large quantities of vegetable matter are undergoing decomposition.

Among some of the wood-stave pipes which have proven highly successful are those of the Denver Union Water Company. The soil in which these pipes are laid is only slightly alkaline with

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 65.

chloride and sulphate of sodium, there being no carbonate of sodium. The pipes are also under a heavy pressure, which is favorable to long life by maintaining complete saturation in the wood. Mr. Finkle.

These matters are referred to merely for the purpose of showing the grave responsibility resting on an engineer when he selects materials for use in construction work.

The discussion by Professor Carpenter on alkali is also a matter of much importance to an irrigation engineer. It does not necessarily follow that all land containing alkali, even in considerable quantities, is useless. The most common forms of alkali in California are sulphate of sodium and chloride of sodium. These are not harmful to the growth of many crops, and some such alkaline land, as for example in the great Imperial Valley, is extremely productive and valuable; but it is important to know that it is necessary to cultivate such land with intelligence, after it is brought under irrigation. Instances have been observed where such land has been heavily fertilized with manure, or where it has been plowed, turning under large amounts of vegetable matter, and then irrigated. Invariably, this results in fermentation and the production of carbonic acid gas in the soil. As carbon dioxide has a greater affinity for sodium than either sulphuric acid gas or chlorine, the sulphates and chlorides of sodium are converted into carbonate of sodium, which is a very destructive alkali and renders the land worthless.

Alkali land which may thus be impaired should be cultivated with as little humus for fertilizing it as possible, and should not be oversaturated with irrigation water, but kept as dry as possible, consistent with the production of crops, and must at all times be well cultivated in order to aerate the soil.

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PAPERS AND DISCUSSIONS

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CURVE RESISTANCE IN WATER PIPES.
Discussion.*

By George Jacob Davis, Jr., Assoc. M. Am. Soc. C. E.

Mr. Davis. George Jacob Davis, Jr., Assoc. M. Am. Soc. C. E. (by letter).—During the past six months some experiments on curve resistance in water pipes have been made, under the writer's direction, in the Hydraulic Laboratory of the University of Wisconsin. A description of these experiments and a discussion of the results are given in the hope of throwing some additional light on the subject.

The experiments were first undertaken in the spring by Messrs. H. Hosler, C. M. Kehr, and R. K. McComb as a graduating thesis in the College of Mechanics and Engineering. They were continued during the summer school session by Messrs. A. F. Coleman, C. M. Kehr, and B. M. Reynolds. Credit is due these gentlemen for the diligent care exercised in making the observations and most of the computations.

The apparatus, which forms part of the equipment used in the regular course of instruction in experimental hydraulics, is shown in Fig. 14. The pipes, marked "Pipe No. 1" and "Pipe No. 2," were lap-welded pipes of iron or steel, and were new at the beginning of the work. The nominal size was 2 in., but the actual internal diameter was $2\frac{1}{16}$ in. In all computations of velocities, ratios of radius to diameter, etc., the actual diameter was used. The piezometers, as shown in Fig. 14, consisted of four $\frac{1}{4}$-in. holes drilled through the pipe

*This discussion (of the paper by Ernest W. Schoder, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for May, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

walls, 90° apart, and communicating with an equalizing chamber, which led to the gauges, and from which accumulated air could be allowed to escape. Water differential gauges were used to measure the loss of head. The water was measured in a galvanized-iron tank of 50 cu. ft. capacity, and the time was observed with a stop-watch. During the spring experiments the length of pipe between the drum and the first piezometer, marked *"l"* on Fig. 14, was only 9 ft., and while calibrating Pipe No. 1 this distance was only 3 ft. It was believed that normal flow in the pipe had not obtained in so short a distance from the rounded entrance to the pipe, therefore, in the summer experiments, the length, *"l,"* was made 62 ft. Mr. Davis.

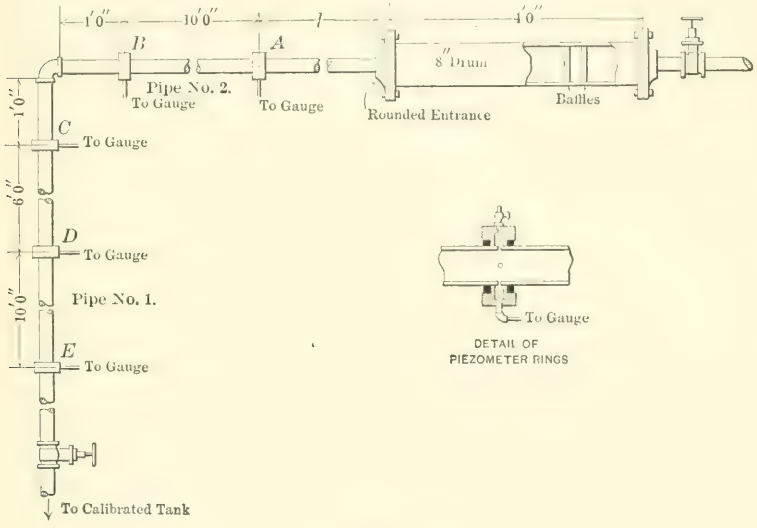

FIG. 14.

The loss of head due to friction in the straight pipes was determined for both pipes before the baffles, shown in Fig. 14, were placed in the drum. A second determination was made for Pipe No. 2 after installing the baffles, and no change was noticed in the friction loss, but the gauges were steadier. At the completion of the spring work, the friction loss was again determined for both pipes. Only a slight difference was noticed, due probably to rusting of the pipes. At the completion of the summer work, the friction loss was again determined for both pipes. This determination gave a loss of head per foot length of pipe equal to

$$h = 0.00240\ v^{1.85}$$

The loss per foot was the same in both pipes. The results of the friction loss determinations are plotted logarithmically in Fig. 15.

Mr. Davis. Only the curve representing the last determinations was drawn. As may be seen from Fig. 15, a line through the points of the earlier determinations would have had a flatter slope and would have lain above the one drawn, up to a velocity of about 10 ft. per sec., at which point the lines cross. It is believed that the effect of rusting would have raised the line instead of lowering it, and the lower values of the friction loss are believed to be due to a steadier and more normal flow at the greater distance from the drum. All the curves having a ratio of radius of curvature to diameter of pipe equal to or greater than $2\frac{1}{2}$ were experimented on under the improved conditions; moreover, during all measurements of loss due to elbows, the nearest piezometer was 19 ft. from the drum. It was thought, therefore, that the nearest estimate of the friction loss was obtained by using the curve drawn in Fig. 15. Any errors due to uncertainty regarding the value of the pipe friction would be small, and would only affect the short-radius curves, the losses in which are large.

All the curves experimented on made a turn of 90°, and had radii of curvature as given in Table 11.

TABLE 11.

No.	Description.	Internal diameter of elbow, in inches.	$\frac{R}{d}$
1	Malleable-iron elbow	$2\frac{3}{8}$	0.788
2	Cast-iron, long-turn, drainage elbow	$2\frac{1}{16}$	1.150
3	Cast-iron, drainage elbow	$2\frac{1}{16}$	0.728
4	Cast-iron, short turn	$2\frac{3}{8}$	0.728
5	Cast-iron, long sweep	$2\frac{3}{8}$	1.520
6	Cast-iron tee	$2\frac{1}{16}$	0
7	Cast-iron	$2\frac{1}{16}$	2.500
8	Wrought-iron or steel	$2\frac{1}{16}$	5.000
9	" " " "	$2\frac{1}{16}$	10.000

Nos. 1 to 6, inclusive, were standard fittings purchased from the Crane Company. No. 7 was cast from a pattern made in the shop of the mechanician of the Engineering College. Nos. 9 and 10 were bent from 2-in. pipe by the University blacksmith.

Drawings of Elbows Nos. 1 to 6, inclusive, are shown in Fig. 16. No. 7 was similar to Nos. 2 and 3; the only difference being in the longer radius of curvature. Nos. 8 and 9 were fitted with ordinary flanges, no attempt being made to obtain a flush joint.

While experimenting on the loss due to the curves, the loss of head was read by three separate water differential gauges; the first being connected between Piezometers *B* and *C* (Fig. 14); the second between Piezometers *C* and *D*, and the third between Piezometers *D* and *E*. The results show that the curves caused a great increase in

Mr. Davis.

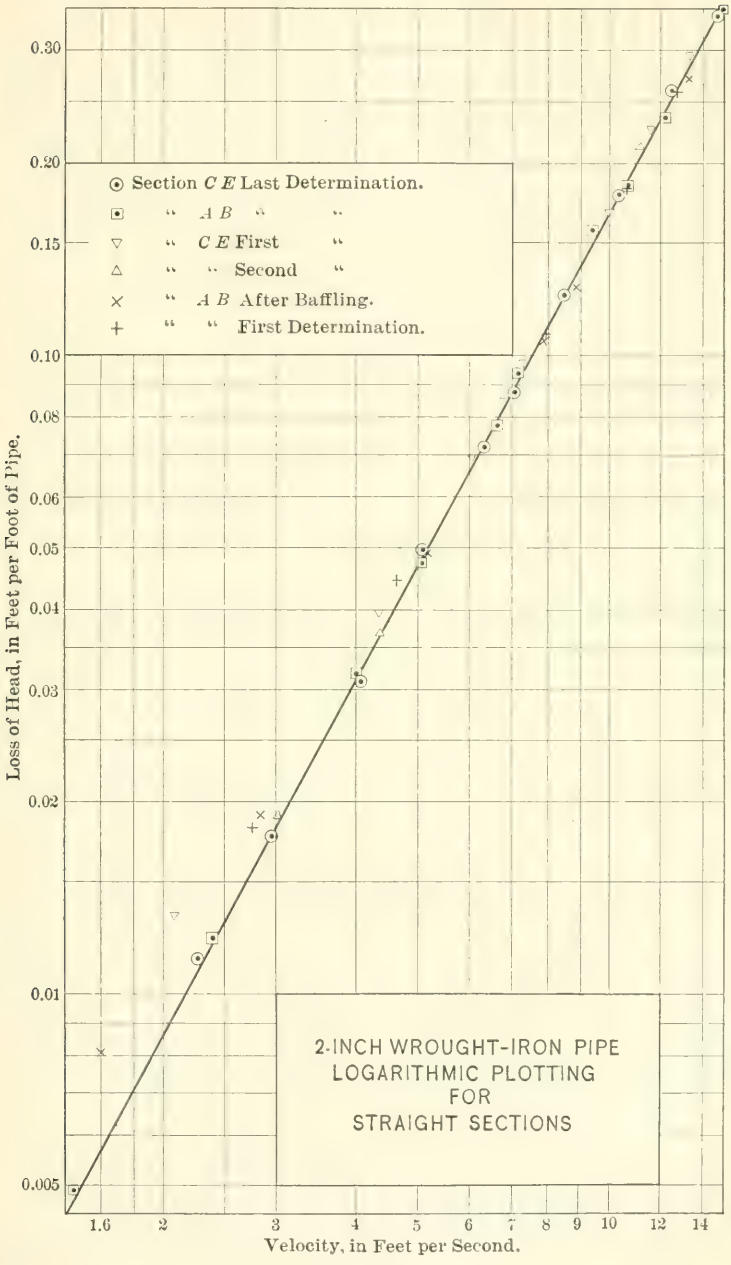

FIG. 15.

Mr. Davis. loss of head in Sections *B-C* and *C-D*, but none in Section *D-E*, even in the case of the tee which caused the greatest total loss of head. Therefore, it is safe to conclude that, by reading the head between Piezometers *B* and *E*, the entire loss caused by the curves was included. The results were all worked up on this basis.

A summary of the data is given in Table 12. Each value of v, which represents the actual mean velocity in the pipe, is the average for several runs taken under identical conditions, except for very slight variations in flow due to pressure changes in the supply mains. Likewise, the value of H_1, which represents the loss of head between Piezometers *B* and *E*, is the average for several runs, the gauges having been read from five to ten or more times during each run.

Discussion of Results.

In discussing the results of experiments on curve resistance, confusion may arise on account of the many ways in which the problem may be considered. Professor Schoder has presented his results worked up on two different bases. The results of the writer's work were studied on five different bases, and the results are compared in curves *A*, *B*, *C*, *D* and *E*, in Figs. 18 and 19.

Case A.—In Case *A* the writer has assumed that a straight pipe, the friction factor of which is already known, is to be bent into a curve of some given radius, and it is desired to know how much more the loss of head will be than in the straight condition. This is the basis of comparison used by the author in Figs. 3 and 4 and the plotting of Mr. Brightmore's experiments in Fig. 13. This case may have a practical bearing in connection with such flexible pipes as fire hose. In this case the desired data were secured by deducting from the total loss of head, as shown by the gauge readings between Piezometers *B* and *E*, such an amount as would be obtained by multiplying the actual center-line length, measured along the curve between *B* and *E*, by the loss of head per foot of straight pipe at the given velocity, as taken from Fig. 15. The results of such computations are plotted on logarithmic cross-section paper in Fig. 17. It will be seen that such a plotting gives a straight line for each pipe curve, at least for velocities greater than 2 ft. per sec. Unfortunately, the lines are not parallel, which indicates that the radius of curvature of the bend, expressed in terms of pipe diameters, has some effect on the exponent of v in a formula of the form

$$H_2 = m\ v^n.$$

where H_2 = the excess loss of head as considered in this case. From Fig. 17, the curves marked *A*, in Figs. 18 and 19, have been prepared by reading off the values of H_2 where the various lines cross the v lines for $v = 15$ and $v = 5$ ft. per sec. By an inspection of Curve *A* it will

Mr. Davis

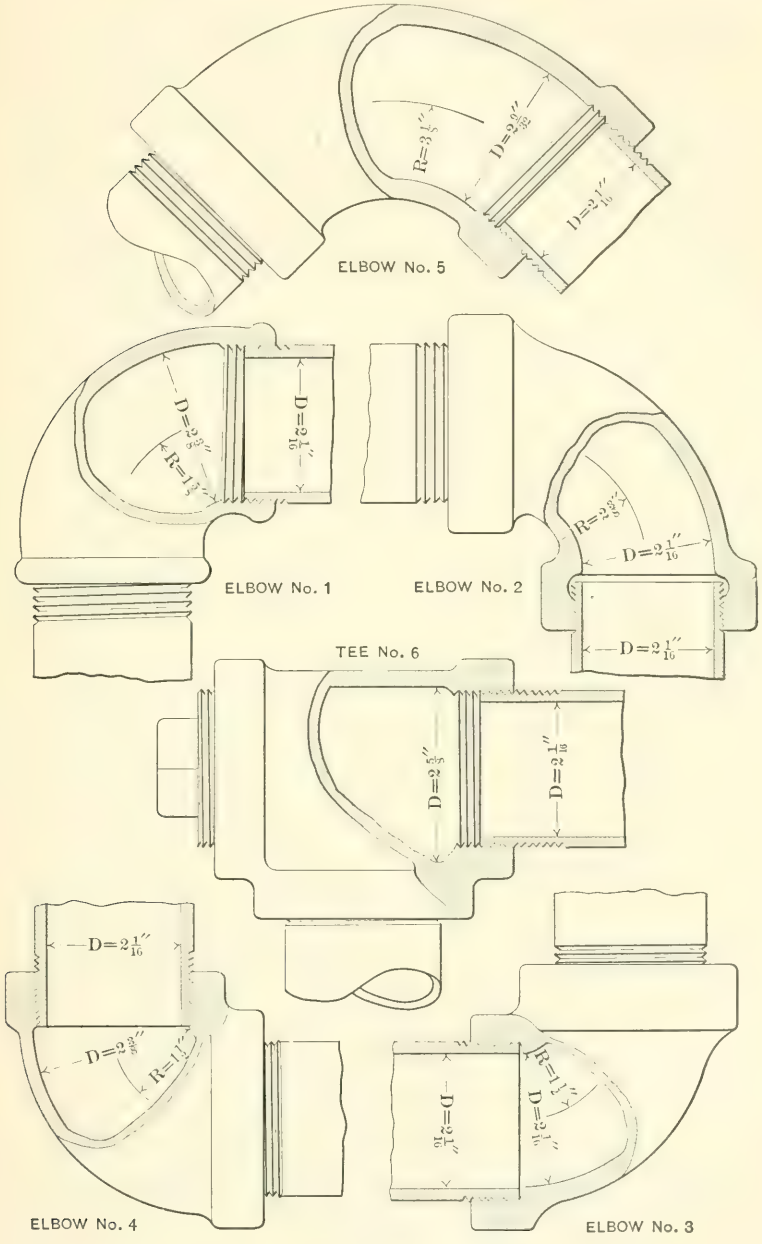

FIG. 16.

TABLE 12.—Summary of Data.

Pipe No. 1.

First Determination of Friction.		Second Determination of Friction.		Last Determination of Friction.	
Velocity, in feet per second.	Loss of head, in feet per foot of pipe.	Velocity, in feet per second.	Loss of head, in feet per foot of pipe.	Velocity, in feet per second.	Loss of head, in feet per foot of pipe.
2.087	0.0133	3.02	0.0189	2.25	0.01133
4.30	0.0396	4.35	0.0365	2.95	0.01762
6.08	0.0592	6.85	0.0863	4.05	0.03077
7.31	0.0968	11.20	0.2110	5.07	0.04940
9.90	0.1667	.....	.....	6.34	0.07188
11.60	0.2240	.....	.....	7.05	0.0876
13.40	0.2936	.....	.....	8.48	0.1235
.....	.....	.....	.....	10.28	0.1762
.....	.....	.....	.....	12.45	0.2585
.....	.....	.....	.....	14.70	0.3388

Pipe No. 2.

First Determination of Friction.		Determination of Friction after Baffling.		Last Determination of Friction.	
Velocity, in feet per second.	Loss of head, in feet per foot of pipe.	Velocity, in feet per second.	Loss of head, in feet per foot of pipe.	Velocity, in feet per second.	Loss of head, in feet per foot of pipe.
2.75	0.0182	1.60	0.0081	1.43	0.0049
4.64	0.0441	2.84	0.0190	2.38	0.0122
7.93	0.1075	5.14	0.0490	4.00	0.0317
10.57	0.1821	7.85	0.1058	5.05	0.0471
12.60	0.2560	8.84	0.1272	6.64	0.0774
.....	.....	13.30	0.2690	7.17	0.0935
.....	.....	.....	.....	9.33	0.1563
.....	.....	.....	.....	10.57	0.1840
.....	.....	.....	.....	12.03	0.2324
.....	.....	.....	.....	14.79	0.3455

Elbow No. 2.		Elbow No. 3.		Elbow No. 7.	
Velocity, in feet per second.	Loss of head between *B* and *E*.	Velocity, in feet per second.	Loss of head between *B* and *E*.	Velocity, in feet per second.	Loss of head, between *B* and *E*.
1.86	0.1887	2.72	0.3681	14.925	7.7775
3.75	0.5960	3.77	0.6496	12.233	5.3279
5.01	1.1848	4.90	1.0435	10.695	4.2053
6.72	2.0407	5.06	1.1490	8.8675	2.8812
8.47	2.8657	6.72	1.9430	7.565	2.1200
9.99	3.9210	8.77	3.1200	6.590	1.6239
11.73	5.4280	9.96	4.0370	5.525	1.1796
14.59	7.6684	10.99	4.9187	3.600	0.5200
.....	.....	13.53	7.2370	3.000	0.3730
.....	.....	.....	.....	2.195	0.2009

Elbow No. 8.		Elbow No. 9.		Tee.	
Velocity, in feet per second.	Loss of head between *B* and *E*.	Velocity, in feet per second.	Loss of head between *B* and *E*.	Velocity, in feet per second.	Loss of head between *B* and *E*.
1.75	0.1512	1.67	0.1552	1.400	0.1348
2.34	0.2476	2.14	0.2334	1.957	0.2446
3.31	0.4785	3.27	0.5107	3.180	0.6236
5.22	1.1258	4.48	0.6206	4.195	1.0368
6.72	1.8214	6.52	1.8597	5.787	1.9041
7.37	2.1516	7.473	2.3841	6.880	2.7099
8.89	3.1102	8.882	3.3344	8.355	3.8200
10.62	4.3664	10.610	4.7400	10.106	5.5910
12.36	5.8523	12.402	6.3570	11.160	6.7250
14.795	8.0186	14.364	8.4408	12.990	9.0530

Mr. Davis.

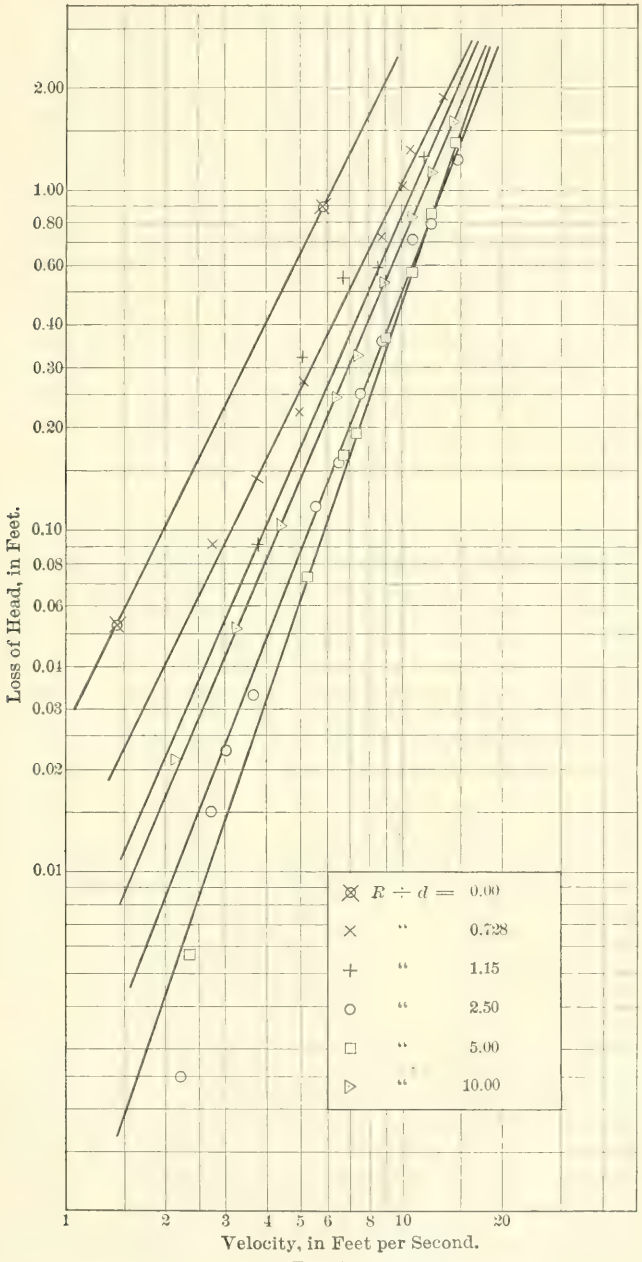

FIG. 17.

Mr. Davis. be seen that a right-angled turn, such as a tee, gives a loss of head of 5.6 ft. under a 15-ft. velocity. As the radius of curvature, R, increases from the value 0, given by the tee, the loss of head at the given velocity very rapidly decreases, reaching a minimum when R is approximately equal to four diameters of the pipe. The curve then apparently rises to a maximum at about eight diameters, and then probably drops off again gradually. Unfortunately, the pipes tested did not have the proper radii of curvature to fix absolutely the minimum and maximum points, nor any points beyond $R = 10d$, and the portions of the curves to the right of the latter point have been merely

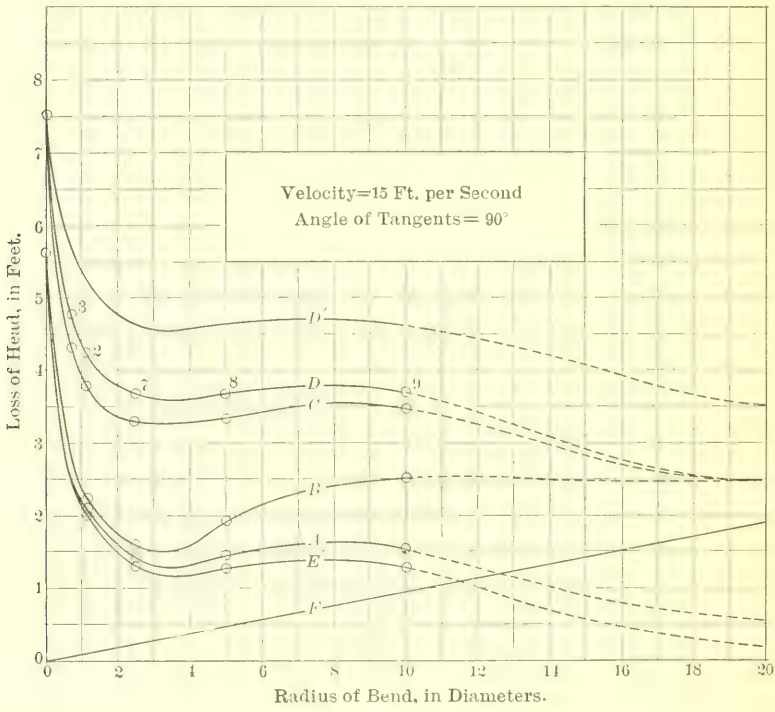

FIG. 18.

sketched in, in accordance with the writer's estimation of their direction, as based on Mr. Brightmore's curves shown in Fig. 13. Perhaps these curves should rise more, as indicated by the Detroit experiments.

Case B.—It may at times be desired to know the loss of head occasioned by a curve in a pipe line, without regard to its length or what effect a variation in its length may have on the total friction loss of the system. It is not possible to measure this loss by piezometers placed at the two extremities of the curve, because a considerable part of the curve loss occurs in the down-stream tangent. A curve dis-

charging into the air at its down-stream extremity will not develop the resistance that it would with a piece of straight pipe beyond; but, by observing the actual loss between two points, as B and E in Fig. 14, and deducting from it the loss normally occurring in a length of straight pipe equal to the length of straight pipe actually existing between B and E, the loss, as defined in this case, will be obtained. Mr. Davis.

The loss of head for this case is plotted against the velocities on logarithmic cross-section paper in Fig. 20. It will be noted that the lines for the curves of different radii are more nearly parallel than those of Case A. From Fig. 20, the curves marked B, in Figs. 18 and 19, have been prepared in the same manner that Curve A was prepared from Fig. 17. It will be remembered, from the definitions, that Curve B gives the total loss, while Curve A gives only the excess loss over what would occur in an equal length of straight pipe. The

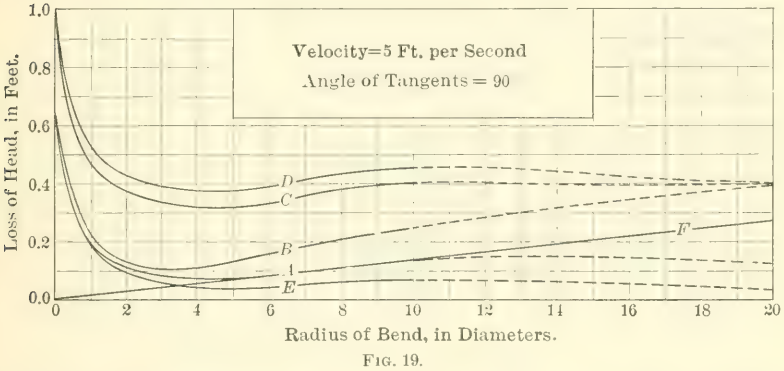

FIG. 19.

vertical intercepts between Curves A and B, therefore, must be equal to the loss of head in a length of straight pipe equal to the length of the curve The straight lines, F, in Figs. 18 and 19, give the friction loss at the respective velocities of 15 ft. per sec., and 5 ft. per sec. for various lengths of pipe expressed in terms of diameters of pipe. The ordinates to this straight line are equal at all points to the vertical intercepts between Curves A and B. Curve B will never fall below the straight line, F, for it does not seem reasonable to suppose that the loss in any curve can be less than the loss in an equal length of straight pipe. If Curve A has been drawn too low or too high, beyond the point where $R = 10d$, then Curve B is also too high or too low by an equal amount.

This case is mentioned by Professor Schoder as one in which the various curves are introduced between two fixed lengths of pipe. It has not much practical value, but is discussed here because it shows the relation between Curve A and the other curves to be described.

Mr. Davis.

Case C.—In this case will be considered the losses occurring in pipes consisting of a straight part and a curve, the total lengths to be the same in all, but the lengths of straight part to vary inversely as the length of curve. The amount of straight pipe to be included must be chosen arbitrarily. The writer has used a length equal to 20 pipe diameters when the length of the curve is zero. Then, when the length of the curve is equal to 20 pipe diameters, the length of straight pipe included will equal zero. Curves *C* on Figs. 18 and 19 were most easily plotted by simply adding to the ordinates of Curve *A* an amount equal to the ordinate of Curve *F* at the 20*d* point. That is, Curve *C* represents the sum of the loss due to friction in 20 diameters of straight pipe, plus the excess loss in the curve, over the loss in an equivalent length of straight pipe. Curve *C* might have been plotted also by adding to the ordinates of Curve *B* the friction loss due to the various lengths of straight pipe required to make the total length of pipe equal 20 diameters. These lengths are given in Table 13.

TABLE 13.—Lengths of Curve and Straight Pipe when the Total Length = 20 Diameters.

$\frac{R}{d}$	Length of curve, in feet, for a $2\frac{1}{16}$-in. pipe.	Length of straight pipe, in feet.
0.0	0.0	5.41
0.728	0.20	5.21
1.15	0.31	5.10
2.50	0.68	4.73
5.	1.35	4.06
10.	2.71	2.70
20.	5.41	0.0

Case D.—Of more practical importance than any of the cases thus far discussed is the case where two fixed points are to be connected by two straight pipes joined by a 90° curve, and it is desired to know the relative total losses which will occur between the fixed points with different curves. Here it is necessary, arbitrarily, to assume some fixed points. In the computations the writer has chosen points that would lie at the extremities of a curve having a radius of 20 diameters. For this curve there would be no straight pipe, but the loss due to the curve, occurring in the down-stream tangent beyond the point considered, has been taken account of. When the radius of curvature is zero, the total length of the two straight pipes would be 6.88 ft. for a $2\frac{1}{16}$-in. pipe. Curve *D* represents the losses for this case. The curve can be constructed by adding to the ordinates of Curve *A* the product of the loss per foot in the straight pipe multiplied by the length of straight pipe required to connect the given points. These lengths are given in Table 14.

Mr. Davis.

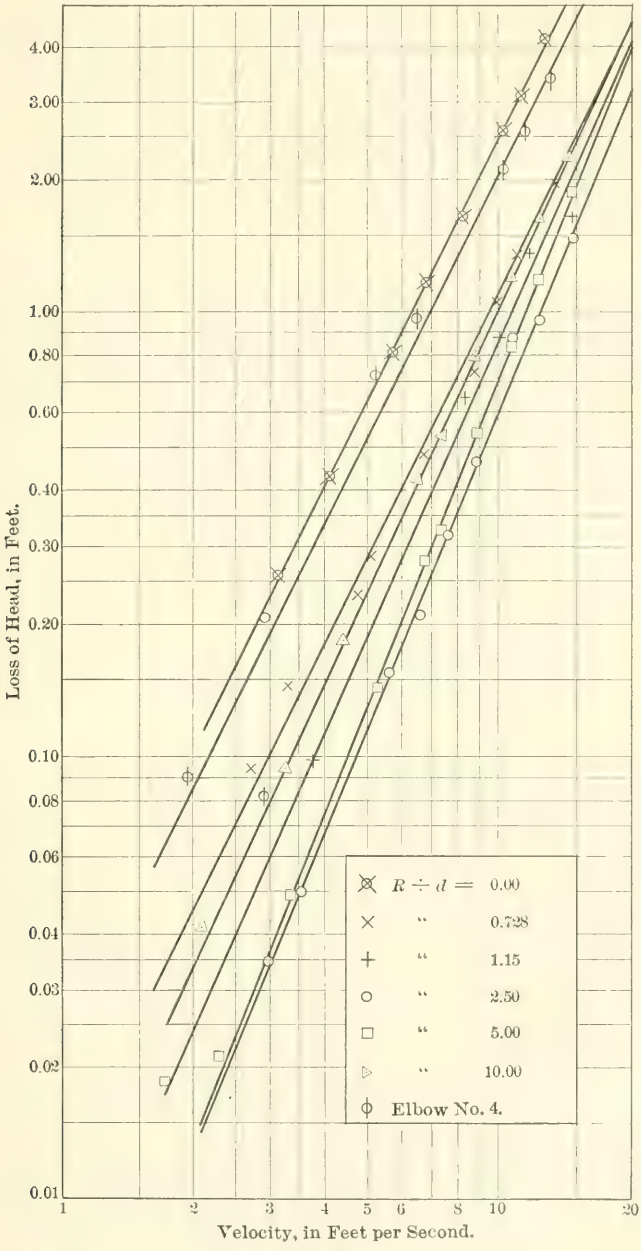

FIG. 20.

Mr. Davis.

TABLE 14.

$\frac{R}{d}$	Length of curve, in feet, for a $2\frac{1}{2}$ in. pipe.	Length of straight pipe, in feet.
0.0	0.00	6.88
0.728	0.20	6.63
1.15	0.31	6.48
2.50	0.68	6.02
5.00	1.35	5.16
10.00	2.71	3.44
20.00	5.41	0.00

Owing to the arbitrarily assumed position of the fixed points, Curve D will meet Curves C and B at the $20d$ ordinate. If some other position of the fixed points had been chosen, it would simply shift Curve D an amount depending on the length of additional straight pipe introduced between the fixed points. For example, Curve D' has been drawn for the loss of head between two points located so that a 90° turn having a radius of 30 pipe diameters would connect them.

Case E.—None of the curves, A, B, C, or D, is very convenient to use in estimating the loss of head in a given pipe system containing bends. Curve E has been plotted for more convenient use.

In Fig. 21, let x represent the loss of head which would occur in a length of straight pipe equal to $AB + BC$; let y represent the loss which would occur in a length of straight pipe equal to $DB + BE$, and let z represent the actual loss caused by the curve, DE, as given by Curve B on Fig. 18.

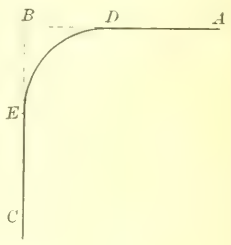

FIG. 21.

Then the actual loss of head occurring between A and C would be given by

$$x - y + z.$$

For the case of the tee, y becomes zero. For curves of various radii, x in the above equation remains constant, while y increases with the radius of curvature, and z follows the law illustrated by Curve B of Fig. 18. Since x is a constant for all cases, the points, A and C, can be most conveniently chosen so that $x = 0$. Then Curve E, Fig. 18, can be plotted, using $z - y$ as ordinates. The distance, DBE, is equal to 1.27 times the length of the curve, hence it is likely that, for bends of large radius, the loss, y, may be greater than the loss, z, and Curve E may then have negative ordinates. This Curve E corresponds to the author's curves in his Figs. 5 and 6.

To estimate the total loss of head in a pipe system, therefore, one should compute the loss in the straight pipe as though it ran directly to the point of intersection of the tangents and to this loss add the loss

given by Curve E for the particular bend to be used. Of course, an E curve must be computed for each velocity. The data for Curve E, for a velocity of 15 ft. per sec., are given in Table 15. Mr. Davis.

TABLE 15.

$\frac{R}{d}$	Length of tangents, DBE.	Loss in curve.	Loss in tangents.	Net loss.
0.0	0.0	5.6	0.0	5.60
0.728	0.25	2.5	0.09	2.41
1.15	0.396	2.2	0.15	2.05
2.50	0.86	1.6	0.3	1.30
5.00	1.72	1.9	0.63	1.27
10.00	3.44	2.5	1.20	1.30

The Effect of Pipe Diameter.

In Fig. 22 a comparison is made of the loss of head occasioned in pipes of different sizes by curves having a ratio of $R \div d$ equal to 4, in all cases, for velocities of 5 ft. per sec, and 10 ft. per sec. Points A are taken from Curves A in Figs. 18 and 19. Points B and C are from the author's plotting of Mr. Brightmore's work in Fig. 13. Points D

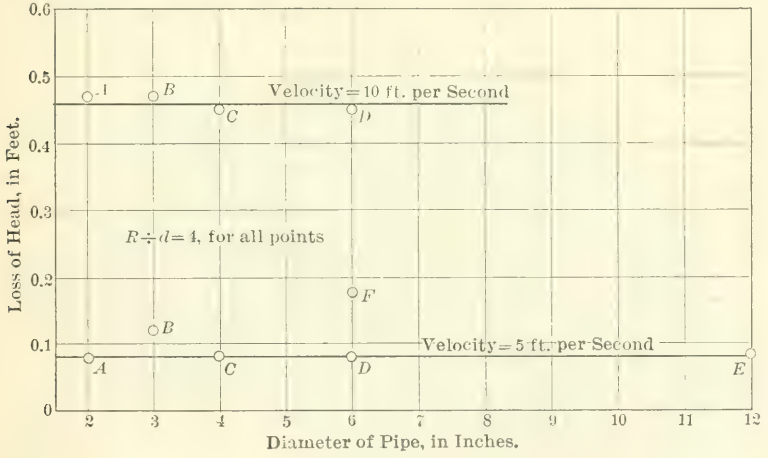

Fig. 22.

are from the author's curves in Fig. 4. Point E is from Table 36 of the Detroit experiments.* These points seem to indicate that, for a radius of curve equal to 4 pipe diameters, the loss of head is independent of the size of pipe. An inspection of the data seems to indicate that the law holds for other ratios of R to d. Point D, it will be

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 181.

Mr. Davis. noted, is based on the author's first straight-pipe experiments. Point F is based on his second straight-pipe experiments. The fact that it does not lie on the line for a velocity of 10 ft. per sec., where it belongs, leads the writer to think that the author's second straight-pipe experiments should not be used, but that the first experiments give reliable results.

Magnitude of Losses.

The author has given an indication of the magnitude of the losses by computing the lengths of straight pipe giving losses equal to those caused by the various curves at different velocities. The writer wishes to show the actual horse-power consumed in pumping water against the curve resistance. The horse-power may be obtained by using the formula

$$h\,p = \frac{H_E \times v \times A \times 62.5}{550}$$

in which H_E = the head lost, as given by the methods of Case E,
v = the mean velocity, in feet per second,
A = the area of cross-section of the pipe, in square feet.

By this formula, curves were plotted for the 2-in. pipe of these experiments. The total horse-power lost, due to a bend in a pipe of this size, is necessarily small, owing to the small weights of water discharged. The writer, therefore, has made use of his assumption that the loss of head is independent of the size of pipe, and has enlarged the horse-power scale to give the losses in a 12-in. pipe, by simply multiplying by $\frac{12 \times 12}{2 \times 2}$, since the weight discharged is proportional to the square of the diameter of the pipe. Therefore, Fig. 23, prepared in this way, may be used for any size of pipe by constructing a suitable horse-power scale.

It is the writer's intention, at the first opportunity, to continue the experiments on the 2-in. pipe, in an attempt to fix more definitely the direction of the curves for bends of long radius, and the results of the further study will constitute the basis of a Bulletin to be published by the University of Wisconsin.

The Effect of Enlargements.

Referring to Table 11, it will be seen that Curves 2, 3, 7, 8, and 9 had the same internal diameter as the pipe, while the others were slightly larger. In the foregoing discussion only the curves having the same diameter as the pipe have been considered. The points on Curve B, Fig. 18, have been numbered to correspond with the numbers of the elbows. It was found that Elbow No. 1 gave the same loss of head as Elbow No. 3. The increase in loss to be expected in Elbow No. 1, due to the slight enlargement and contraction of section, seems to

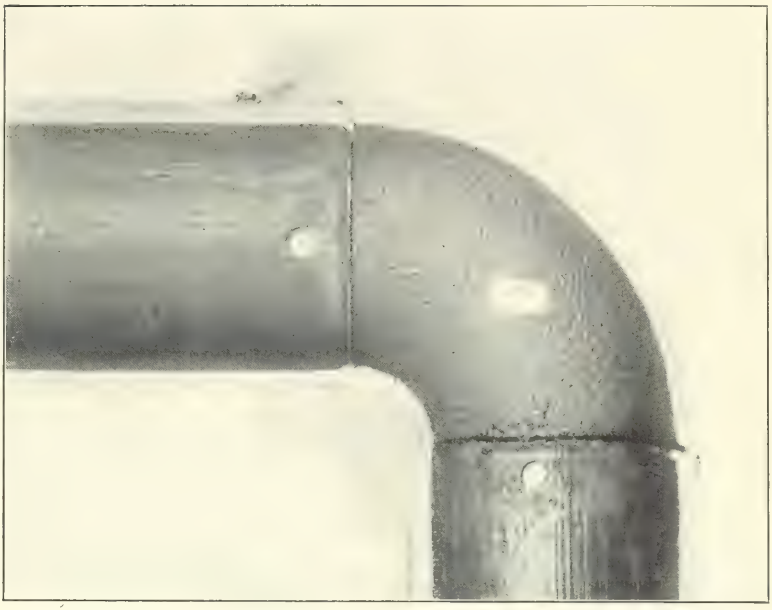

FIG. 1.—INTERIOR OF CAST-IRON "DRAINAGE" ELBOW, NO. 3.

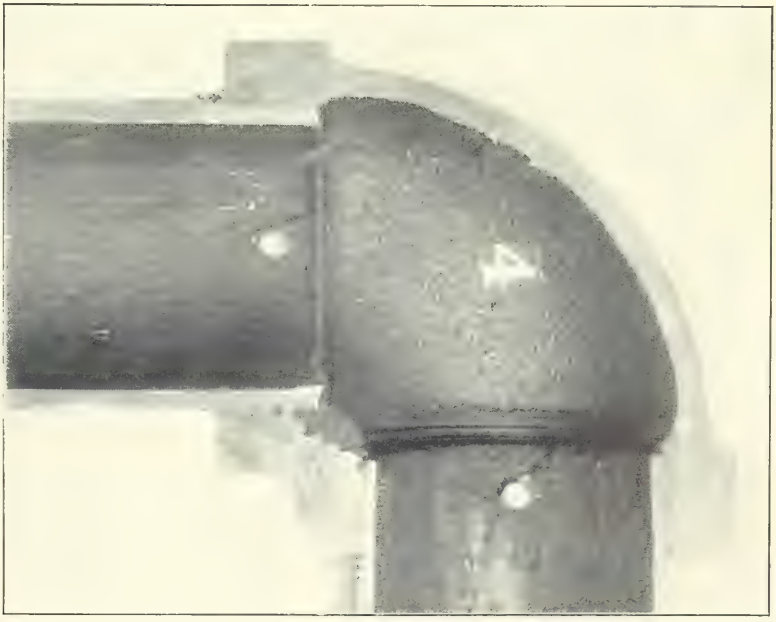

FIG. 2.—INTERIOR OF CAST-IRON "SHORT TURN" ELBOW, NO. 4.

have been exactly balanced by the reduction in loss due to a slightly longer radius of curvature. Mr. Davis.

An idea of the increase in loss due to the enlargement from $2\frac{1}{16}$ in. to $2\frac{3}{8}$ in. and again contracting to the smaller size may be had by comparing Elbows Nos. 3 and 4 which are both of cast iron, of about the same degree of roughness, as may be seen in Figs. 1 and 2, Plate CXIV, and both have a radius at the center line of 0.728 diameter. The pieces of pipe shown riveted to the sections of the elbows are not the pipes used in the experiments, but were inserted merely to show the

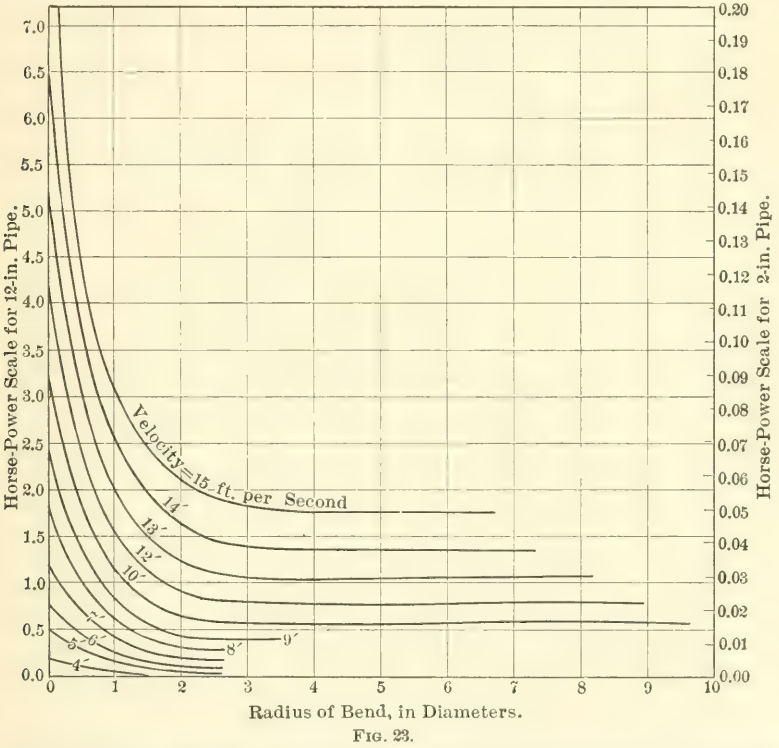

FIG. 23.

actual distance that the pipes were screwed into the elbows. The pipes used in the experiment were used full length and had no burr at the end, like those shown, which was caused by hand cutting.

On the basis of Case *B*, the loss of head for Elbow No. 3 is given by

$$H_3 = 0.0113v^2,$$

and the loss of head for Elbow No. 4 is given by

$$H_4 = 0.0202v^2.$$

The loss in Elbow No. 4, therefore, is about 1.8 times the loss in Elbow No. 3.

Elbow No. 5 gave about the same loss as Elbow No. 2.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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NOTES UPON DOCKS AND HARBORS.

Discussion.*

By Messrs. W. B. Ruggles and E. P. Goodrich.

Mr. Ruggles.

W. B. Ruggles, M. Am. Soc. C. E. (by letter).—If, as the author says, there is a "striking difference" between the plans of the port work of the United States and those of European countries, there is quite as marked a difference of methods between the ports of the States and those of Latin-American countries.

Briefly, the prevailing custom south of the United States is to handle all freight to and from steamers with lighters. To some extent—but not entirely—this is due to a lack of good, deep-water harbors. There are few satisfactory harbors on the Pacific Coast of North and South America south of San Francisco. The peninsula of Lower California gives some protection to the few ports along the Gulf of California, but the deep-draft vessels going north from Panama, with sixteen regular ports, can only dock at Corinto in Nicaragua, Manzanilla in Mexico, lately completed, and Salina Cruz, at the western terminus of the lately constructed Tehuantepec Railway, in Lower Mexico. The latter port has been supplied by the Mexican Government with modern harbor facilities, and the railway is expected to compete sharply for transcontinental business. Steamers going south from Panama, with forty-five scheduled ports, find good piers only at two, Callao and Valparaiso.

On the Caribbean Sea, vessels coasting north find adequate docking

*This discussion (of the paper by Luther Wagoner, M. Am. Soc. C. E., printed in *Proceedings* for May, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

facilities at Port Limon, and others are being provided at Puerto Bar- Mr. Ruggles.
rios; but these are reported to be in comparatively shallow water. The northerly or Gulf port of the Tehuantepec Railway (Mexico) has the lately completed pier of Coatzacoalcos. Tampico has a Government steel pier and custom-house. Vera Cruz has a stone pier and a protected harbor 33 ft. deep. Atlantic steamers on their south-bound voyages can dock at Barranquilla and Cartagena.

There are a number of piers on the north coast of Venezuela, and, beyond Venezuela, on the east coast of South America, there are good harbors at long intervals.

The Island of Cuba is especially fortunate in its land-locked, deep-water harbors, and yet the bulk of the cargoes to and from foreign ports is handled by lighters, some coastwise sailing vessels being of such light draft as to reach the small landing piers usually found in shallow water. Havana Harbor is ideal; that bay, the Harbor of Cienfuegos—6 miles long and of good depth—and the Harbor of Santiago, with another 6-mile bay, are all as closely land-locked as they could be and give good entrances. Santiago has a pier for the Juragua Iron Company and a commercial pier, but not for vessels of deep draft. Guantanamo, 50 miles from Santiago, is a good harbor under improvement; but, as it has been taken over by the United States as a coaling station, it is hardly to be considered as a Cuban port. Cardenas Bay is moderately well protected, but, except as dredged, is shallow at its entrance. Matanzas Bay, extending 6 to 8 miles inland, is open, but deep—more than 1 000 ft. deep within a mile or so of the old Customs wharf. Lighters are used in all these harbors. Nipe Bay, on the north coast, is now a terminal point for a railroad and no doubt will be improved. Bahia Honda is a magnificent harbor, one of those which it was expected would be taken as a coaling station by the United States, but it has no port town of importance. Barracoa is good, and there are other fair harbors.

Except that in the course of nearly 400 years of existence the City of Havana has so encroached upon the bay as to make any kind of a belt railway very expensive, the harbor offers one of the best opportunities to provide the most modern facilities for the handling of the heavy passenger and freight business of what was once the fourth port of North America. In the ten years dating from the early American occupation of Cuba, plans for such improvements have been perfected, and in some measure approved by the Cuban Government, although, in the late adjustment of affairs, the present Provisional Government has apparently not found it advisable to give these projects precedence over the older and approved contracts for the sanitation of the city. Possibly the results of efforts to provide adequate harbor facilities for Matanzas may, in some measure, have led to this conclusion. Early in the former American occupation of Cuba, the Intervening Government was for-

Mr. Ruggles. mally asked, by the city authorities at that port, to provide suitable harbor accommodations for the Bay of Matanzas, and after a year or so of preparation, in which the local, civil, and commercial interests were frequently consulted, and with no openly declared opposition from any interest, plans and specifications were submitted for a deep-water, creosoted-pile pier in the least exposed part of the bay, having permanently constructed connections with the two railways entering the city. One of the very last acts of the outgoing Intervening Government, after a full review of the situation with, and a hearty consent by, the new Cuban authorities, was the approval of the contract with a reliable New York firm for the construction of the pier, railway, steel bridges and other accessories, at a cost of approximately $300 000; with some subsequent modifications of plans and specifications the works were built in 1902-03, but, in the five years since their completion, they have been used very little, if at all. The attitude of the local owners of the fleets of lighters and of the warehouses, of which Matanzas has an exceptionally substantial and extensive system on, or near, the San Juan River, has been distinctly hostile to the new arrangement, and, so far, they appear to have controlled the situation. It was hoped the new provisions would ultimately lead to more direct shipments from the interior of the island to the vessel. Even if the warehouse and lighter interests are themselves shippers, as it happens they are, it is but natural that they should hesitate to support such a system, for their profits lie principally in handling the sugar, etc., at the port.

In Jamaica, a possession of the very commercial English people, there is an excellent closed harbor and deep-water pier at Kingston, and like facilities at Port Antonio on the north coast. The Port of Spain, in Trinidad, another English harbor, though not the deepest, provides at least for Royal Mail steamers.

The Danish port at St. Thomas has a floating dock. In San Domingo, a pier at Macoris, built under political grant, is reputed to have paid its considerable cost by port duties. The ports of Haiti are noted as "slow and inefficient." In Puerto Rico, San Juan is a dredged harbor, and, with other ports of the island, is being improved since it became an insular possession of the United States.

Before the construction of the Panama Railroad in 1850-55, Puerto Bello, 18 miles below Colon, was the desirable harbor on the Caribbean Sea, well protected and deep; it is still used as a shelter harbor at times of violent and long-continued gales.

The Harbors of Colon and La Boca, near Panama, were not naturally very good, both being originally shallow and, except for the islands in the Pacific, open, but the dredging for the Panama Canal makes their development into good ports reasonably practicable. All vessels can receive and discharge freight at the new wharf, No. 11, at Cristobal—that part of the harbor town within the Canal Zone. The

Mr. Ruggles.

Royal Mail and Hamburg-American lines still have their own piers at Colon, where, to date, they have proved adequate. On the Pacific side, the old French pier at La Boca, with its extensions, lying along the Canal line, provide berthing space for four or more large, deep-draft vessels at a time. These extensions were made about two years ago when it was presumed that the Canal entrance would be moved eastward to the point of Sosa Hill, and the new structures are, consequently, not of as substantial construction as the original French Canal company's pier, which is 960 ft. long, built, 22 years ago, with steel trusses and frames on steel cylinders, 5 m. in diameter, sunk to rock and filled with concrete. As the new train loads of the Panama Railroad are much greater than at the date of the construction of the pier, the two loading tracks have been removed from the trusses to a roofed pile trestle on the easterly side of the old shed, giving a correspondingly greater storage capacity; otherwise the structure is probably good for another generation. The new channel is being dredged to 45 ft. below mean sea level, and when the breakwater, now in course of construction toward Naos Island, is completed, the entrance to the piers will be well protected. The French company provided loading devices on the old pier and a very large crane of 15 tons capacity at the outer end for heavy machinery. On the late extension there are eight new electric cranes of nominally 8 000 lb. capacity—the writer has seen 10 000 lb. handled on a trial. At Cristobal there are no cranes on Wharf 11, but at Pier 14 there is a large Brownhoist cantilever crane for coal, iron, etc.

Provisions are now being made for the construction of permanent piers between No. 11 and No. 14 along the Canal, the channel of which is kept dredged to required depth. Breakwaters are likewise proposed for this bay, and there is now a small dry dock above Pier 14.

Attention has lately been called to the fact that, except at the port of San Francisco and at Bremerton, Wash., there is no dry dock on the Pacific Coast.* This would seem to imply that the timber dry dock at Puget Sound, built in 1896, is not now available. Of the three at San Francisco, the largest is that of the San Francisco Dry Dock Company, 750 by 122 ft. The statement also fails to mention the dry dock at Salina Cruz, which is 600 by 100 ft., with a depth of 31 ft. In view of this fact and that commercial corporations and companies, such as the above and the Mersey Dock and Harbour Board at Liverpool, do not consider their port work complete without a dry dock, almost always a graving dock (one English firm, The Smith's Dock Company, on the Tyne, has six graving docks supplemented with two floating docks), it would appear desirable to have other harbors supplied with these provisions for cleaning and repairing. There is,

**Cassier's Magazine*, March, 1908.

Mr. Ruggles. it is true, a growing use of floating dry docks, but where there are, for instance, such good foundations within easy reach, with ample space and wide range of tide, as can be found between Corozal and the Canal, two or three miles inland from the La Boca piers, the cost of a permanent graving dock of large dimensions—double if necessary—would probably be no greater than for a corresponding provision for floating docks, if the additional repeated dredging for an operating basin, estimated as at least 50 ft. deep, for a dock of 30-ft. draft, shore anchorage works, etc., be added. At the site suggested it happens there could easily be coaling stations and railroad yards.

In late years the English Government has made great extensions in the harbors at which vessels are loaded with coal, notably that of Blyth, where improvements to cost approximately $900 000 are under way, and England, Holland, Denmark, and, to some extent, France, make special provisions for their fishing vessels. At the Boston, Mass., fish wharves this year nearly 1 000 000 lb. of fish were marketed in one day. As for the coaling staithes, or stations, the Pacific Coast may come in time to provide oil stations instead, but it cannot yet be said that there should no longer be any provision made for coaling steamers, for fuel, and for cargo.

Mr. Goodrich. E. P. Goodrich, M. Am. Soc. C. E. (by letter).—Some investigations made by the writer in connection with the Meadow Reclamation and Dock Commission led to the independent discovery, several months before the publication of Mr. Wagoner's paper, of several points brought out therein.

The rate of increase of population of many American cities, and approximately that of the United States as a whole, was found to be in a logarithmic ratio. A similar law holds for the foreign tonnage of many ports, thus providing a fixed relation between tonnage and population, and, in many instances, the growth of manufactures follows the same law. Because of these facts the per capita values of such commercial and industrial items are of great value in predicting probable future requirements of ports and communities.

Accurate plotting of statistics on logarithmic paper of large scale discloses many interesting facts, among which the following may be cited: At each war of any magnitude, the rate of increase of the population of the United States as a whole, and of a majority of its cities, received a check, and a changed rate was established. Commerce and industry are somewhat similarly affected, but to a much greater extent, by each financial panic which is almost literally a war in those lines.

Prediction of future port requirements is almost as complex as predictions concerning their tidal changes, but without as complete data. Railroad extensions and their rates to a certain port and to competing ports, the prompt adoption of improvements in handling facili-

ties at those ports, the effects of natural limitations and advantages, the possibilities of change in the extent of neighboring industries, like fishing, lumber, grain, etc., in localities where these predominate, are some of the factors entering the problem when studying the necessities of such projects as those of Newark or Jamaica Bay, in the vicinity of New York City, or of increases in facilities in Baltimore, New Orleans, or San Francisco. Mr. Goodrich.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES FRANCIS POWELL, M. Am. Soc. C. E.*

DIED JULY 30TH, 1907.

The personal knowledge of the writer concerning General Powell covers only the four years that he was stationed at New London, while the writer was associated with him in the capacity of Assistant Engineer. Owing to General Powell's modesty, he rarely spoke of his previous career—almost never of his services during the Civil War—hence, in the preparation of this memoir, it has been necessary to depend upon official data and information furnished by others for the facts relating to his early life, and the writer wishes to make acknowledgment of the great aid and valuable information given him, especially concerning General Powell's early life and services during the Civil War, by his brother, Captain A. O. Powell, and by Major Harry Taylor, Corps of Engineers, in furnishing authentic records and dates concerning his service as an Officer of the Corps of Engineers. Without the aid of these gentlemen, the writer would have been unable to prepare this memoir.

Charles Francis Powell was born in Jacksonville, Illinois, on August 13th, 1843. His ancestors were Americans and his great-grandfather was a soldier in the Revolutionary War. Born without the advantages of wealth, Charles Francis Powell was, from boyhood, obliged to depend on his own exertions for support and advancement. While quite young he moved to Milwaukee, Wisconsin, where he obtained his early education in the public and private schools. When the Civil War broke out, and before he was eighteen years old, he began his military career, enlisting on May 10th, 1861, as a private in Company B, 5th Wisconsin Volunteer Infantry, and afterward being promoted to Corporal and then to Sergeant Major.

During the Civil War he saw much hard service and fierce fighting, and his war record is a brilliant one. His first service was in the suppression of the bank riot at Milwaukee in the summer of 1861; later in that year he was ordered East, and in September took part in the advance into Virginia. The following year he was with the Army of the Potomac in the Peninsula Campaign and took part in the Siege of Yorktown, the engagement at Lee's Mill, the Battle of Williamsburg, the engagements at Golding's Farm, Garnett's Farm, Savage's Station, and White Oak Swamp, and the Battle of Malvern Hill, all in less than three months. Later in that year he was on the field at the Second

*Memoir prepared by G. E. Verrill, M. Am. Soc. C. E.

Battle of Bull Run and the Battle of Chantilly, but was not actively engaged. In September, 1862, he took part in the Antietam Campaign and on September 17th participated in the bloody Battle of Antietam. In November he was in the advance into Virginia near Harper's Ferry and along the eastern base of the Blue Ridge. In 1863 he was on the Rappahannock opposite Fredericksburg; participated in the Gettysburg Campaign, and, after a forced march, took part in the great Battle of Gettysburg on July 2d, 3d, and 4th. On July 5th he was in an action with the enemy's rear guard; later in the month he was again in the advance into Virginia near Harper's Ferry, and in August took part in the enforcement of the second draft at New York and Albany. This was his last active service during the Civil War.

He was then only about twenty years old, but he was an experienced soldier, and had seen harder service and more of war and actual fighting than falls to the lot of many men who are soldiers all their lives and attain high rank. He had put in more than two years of the hardest sort of service, and had taken part in several of the greatest and most fiercely fought battles of the Civil War or of history, but he came out unscathed, though the writer has heard him say that he had two or more horses shot under him and his cap carried away by a rifle ball.

On September 29th, 1863, he was appointed a cadet at the United States Military Academy by President Lincoln. At this time the number of cadets at West Point was much reduced because of the disaffection of the southern Congressmen and the cadets appointed by them, and to fill these vacancies a number of cadets were appointed by President Lincoln from among deserving young soldiers of the Union Army, their appointments being from southern Congressional Districts which then had no cadets at West Point. Young Powell was among the soldiers thus appointed, his appointment being from a South Carolina District, and was stated to have been for "soldierly courage and ability, faithful and brave conduct and gallantry on the field of battle."

He passed his entrance examinations to West Point successfully, although, up to this time, his opportunities for education had been somewhat meager, and, for the two years and a half, or thereabouts, since he enlisted he must have had to give up study entirely. He was also under the disadvantage of entering after the academic year had begun, and hence was behind his class, but by hard work and perseverance he managed, not only to make up back work and keep up with his class, but to maintain a high stand. During his last year at West Point he met with an accident that nearly cost him his life. At cavalry drill his horse stumbled and fell with him and badly injured his knee; tetanus subsequently developed, and he was for a long time in the hospital, but finally recovered. Although he was left permanently lame, it was not sufficient to incapacitate him for duty. In spite of this

accident, which would have made most men give up entirely, he graduated twelfth in a class of sixty-three and was commissioned, on June 17th, 1867, a Second Lieutenant in the Corps of Engineers. His subsequent promotions in the Corps of Engineers were: First Lieutenant, April 23d, 1869; Captain, June 17th, 1881; Major, January 26th, 1895; Lieutenant Colonel, January 22d, 1904.

His first duty as an Engineer Officer was with the Engineer Battalion at Willets Point, New York, where he served from August, 1867, to May, 1871, as Company Officer, Post Quartermaster and Commissary, and Battalion Quartermaster, in the latter capacity for the last three years of his service at Willets Point. After leaving Willets Point he served as assistant on the geodetic survey of the Northern Lakes for nearly 8 years, and of the Mississippi River for nearly 2 years, to March, 1879, and as assistant to Major George L. Gillespie, Corps of Engineers, from April 11th, 1879, to October 26th, 1881, when he was appointed Engineer of the 13th Light-house District. This latter position he held until April 11th, 1888, at the same time being in charge of various defenses, river and harbor improvements, surveys, etc., in Oregon, Washington, and Idaho. The Cascades Canal and the commencement of the great jetty at the mouth of Columbia River were among the most important works of which he had charge at this time.

From May 31st, 1883, to November 18th, 1890, he was Secretary and Disbursing Officer of the Mississippi River Commission, and was charged with various duties connected with the improvement of that river; and from 1890 to April 27th, 1893, he was at Bismarck, North Dakota, and Sioux City, Iowa, in charge of the improvement of the Missouri River above Sioux City and the Yellowstone River, in Montana and North Dakota. At the close of the above tour of duty he was appointed Engineer Commissioner of the District of Columbia, Washington, D. C., which position he held until March 2d, 1897, when he was ordered to Pittsburg, Pennsylvania, and placed in charge of river and harbor work in that district, which included the improvement of Pittsburg Harbor, the Allegheny River, the Monongahela River slack-water navigation system, and various other engineering works. On January 8th, 1902, he assumed charge of the Connecticut District, comprising all river and harbor improvements in that State, the Pawcatuck River, between Rhode Island and Connecticut, and the defenses at the eastern entrance to Long Island Sound, his headquarters being at New London, Connecticut. Among the more important works in this district were the construction and equipment of various emplacements for modern high-power guns and mortars at the eastern entrance to Long Island Sound, the improvement of the Connecticut, Thames, and Housatonic Rivers, and the harbors of New Haven, New London, and Bridgeport, besides a number of other smaller rivers and harbors.

In the early part of 1906, while still stationed at New London, General Powell was taken with a severe illness, largely or wholly the result of over-work, but, with his extremely strict ideas with regard to duty, he insisted upon still performing his official duties, even while confined to his room and bed. The writer will always remember one occasion in particular when he was directed to report in person to General Powell regarding certain official business, and found him at his home, barely able to sit up, and yet insistent upon talking over the matter in hand and making his decisions, although seized every few minutes with violent fits of coughing and scarcely able to hold a letter in his hand. Several times during his illness he recovered sufficiently so that, although contrary to the advice of his physician, he went to his office, but finally his condition became so serious that he was obliged to give up all official duties, and on March 8th, 1906, he was relieved of charge of the District. His condition had now become so critical that, upon the representations of his physician and family, his retirement was requested under the provisions of Section 1244, U. S. Revised Statutes. Before this was done, however, and on account of his splendid record during the Civil War, he was appointed Brigadier General, U. S. Army, dating from March 31st, 1906; his retirement took place three days later, April 3d, 1906.

After his retirement his health greatly improved for a time, although his family and friends had scarcely dared to hope for it, but the improvement was not permanent, and on July 30th, 1907, he died at St. Paul, Minnesota, where he had moved with his family after his retirement.

To quote from the General Orders of the Chief of Engineers announcing General Powell's death: "All duties entrusted to him were well and faithfully performed." Strict attention to duty and absolute adherence to the law and to his orders were cardinal principles of General Powell's life. Conscientious to the last degree, he always gave to his work the most unremitting personal attention, even to the smallest details, and it was this close personal application to the daily routine and minor details of his work, which most men in similar positions turn over to their assistants, that took much of his time and rendered his duties unusually arduous and confining.

Personally, General Powell was of an extremely kind, courteous, and pleasant disposition; unassuming and modest in manner and appearance, yet always an officer and a gentleman. To his assistants and those with whom he came into official contact he was always courteous, considerate, and approachable; to his family he was devoted.

He was married on May 17th, 1883, at Albany, Oregon, to Margaret Isabelle, daughter of James H. Foster, and is survived by his wife and six children.

General Powell was elected a Member of the American Society of Civil Engineers on October 3d, 1888.

620.6 William P. Morse T

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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October, 1908.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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PAPERS AND DISCUSSIONS

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THE LOW STAGE OF LAKES HURON AND MICHIGAN.*

By C. E. Grunsky, M. Am. Soc. C. E.

Lakes Huron and Michigan, if mean annual water elevation alone is considered, have been at less than the normal stage since 1888. These lakes were abnormally low in 1895 and in 1896. The Board of Engineers on Deep Waterways, in 1900, called attention to the depressed stage of the lake waters, giving the amount of the depression below what was then thought to be the normal at about 1 ft. for the preceding 15 years. At that time this depressed stage was attributed to certain natural and artificial changes that had been made at the head of St. Clair River, which is the outlet from the lakes.

The importance of restoring the lakes to normal elevation and of holding them so high that navigation interests will be fully protected, coupled with the desirability of withdrawing water from the lake system for sanitary and inland navigation purposes in limited, yet not inconsiderable, amounts has prompted the following study of the effect of water storage in Lake Superior upon the water elevation in Lakes Huron and Michigan, and of the causes to which the protracted low stages of the past in the two lower lakes should be ascribed.

The data herein used are taken from the published annual reports of the Chief of Engineers, U. S. Army.†

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

† See particularly the report of E. S. Wheeler, M. Am. Soc. C. E., Assistant Engineer, in Annual Report of the Chief of Engineers, U. S. A., 1903, Part 4, p. 2855.

LAKE SUPERIOR
Elevation of Water Surface
(Mean Annual)
1903 Datum Plane
603 Ft.
602
601
1860
1865
1870
1875
1880
1885
1890
1895
1900
1905
Sec. Ft.
100 000
90 000
80 000
70 000
60 000
50 000
40 000
Discharge of St. Marys River
Water-Yield of Drainage Basin
5-Year Periods

FIG. 1

Fig. 1 and Plate CXV show the mean annual discharge for each of the two rivers, the St. Mary's, which flows from Lake Superior into Lake Huron, and the St. Clair, which, as already stated, carries the outflow from Lakes Huron and Michigan, and also the water-yield of the drainage basins tributary to the lakes, and the mean annual elevations of Lake Superior and of Lakes Huron and Michigan.

With this information, mass-curves of the water-yield of Lake Superior drainage basin could be constructed, and the effect of storage in this lake upon the discharge through its outlet, the St. Mary's River, could be determined.

Some of the conclusions reached from these studies and from the records referred to are here briefly stated.

TABLE 1.—Lake Stages.

Monthly Means for Typical Years. Elevations in Feet Above Sea Level.*

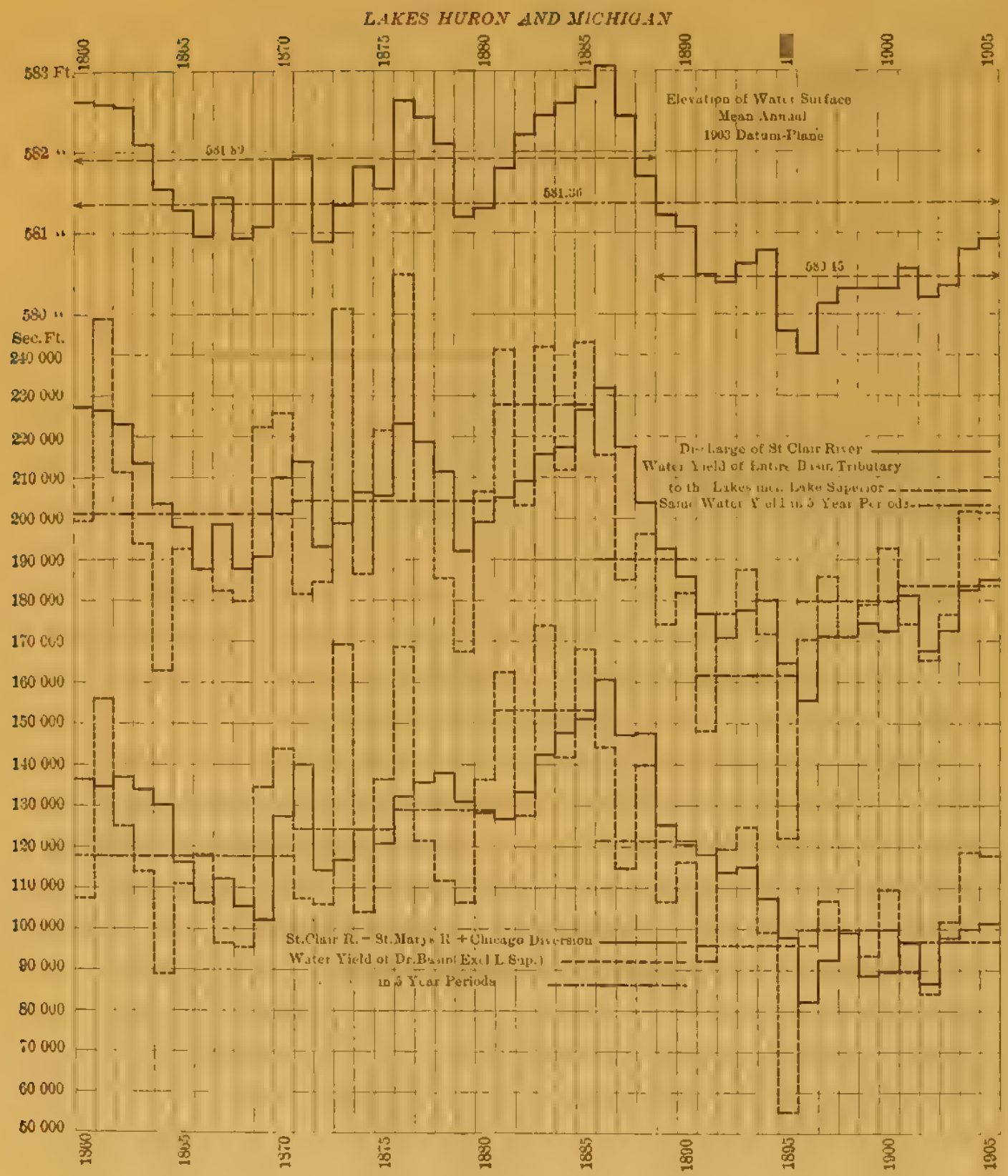
Lake Superior.

Month.	1861.	1869.	1879.	1892.	1901.
Jan.	602.59	602.21	601.39	601.38	602.65
Feb.	602.26	601.97	600.90	601.02	602.38
Mch.	602.12	601.52	600.74	600.84	602.13
Apr.	602.53	602.10	600.87	600.99	602.27
May	603.16	602.50	601.25	601.50	602.65
June	603.31	602.52	601.38	601.86	602.60
July	603.47	602.88	601.68	602.00	602.97
Aug.	603.43	603.34	601.71	602.01	603.19
Sept.	603.34	604.19	601.64	602.07	603.07
Oct.	603.37	603.67	601.66	601.96	603.14
Nov.	603.03	603.33	601.48	601.68	603.08
Dec.	602.65	602.68	601.08	601.40	602.70
Year.	602.94	602.74	601.32	601.56	602.74

Lakes Huron and Michigan.

Month.	1861.	1872.	1876.	1886.	1895.
Jan.	582.83	580.99	581.74	582.67	580.03
Feb.	582.78	580.79	581.72	582.74	579.91
Mch.	582.92	580.29	581.85	582.93	579.92
Apr.	582.89	580.71	582.12	583.22	580.02
May	582.94	581.11	582.73	583.55	580.18
June.	583.18	581.51	583.22	583.64	580.26
July	583.27	581.61	583.66	583.48	580.23
Aug.	583.19	581.58	583.60	583.33	580.14
Sept.	582.99	581.48	583.49	583.15	580.01
Oct.	582.67	581.36	583.09	583.02	579.74
Nov.	582.55	581.06	582.94	582.73	579.33
Dec.	581.20	580.77	582.75	582.43	579.09
Year.	582.86	581.10	582.74	583.07	579.90

*The elevations in Table 1 are not in perfect accord with the lake-stage diagram published by the United States Lake Survey. They are based on elevations published in official reports of 1903.


LAKES HURON AND MICHIGAN
1860
1865
1870
1875
1880
1885
1890
1895
1900
1905
583 Ft.
582 "
581 "
580 "
Sec. Ft.
240 000
230 000
220 000
210 000
200 000
190 000
180 000
170 000
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
Elevation of Water Surface
Mean Annual
1903 Datum-Plane
581.59
581.36
580.45
Discharge of St Clair River
Water Yield of Entire Drain. Tributary
to the Lakes incl. Lake Superior
Same Water Yield in 5 Year Periods
St. Clair R. − St. Marys R. + Chicago Diversion
Water Yield of Dr. Basin (Excl. L. Sup.)
in 5 Year Periods

The area of the drainage basin of Lake Superior is about 76 100 sq. miles; the area of the lake is about 32 100 sq. miles.

The area of the drainage basin of Lakes Huron and Michigan, including the Lake Superior basin, is 213 900 sq. miles, and the water-surface area of the two lower lakes is 45 500 sq. miles.

TABLE 2.—Discharge of St. Mary's and St. Clair Rivers.*

Computed from Published Monthly Means.

Year.	St. Mary's River.		St. Clair River.	
	Elevation, in feet, Lake Superior.	Mean discharge, in second-feet.	Mean elevation, in feet, Lakes Huron and Michigan.	Mean discharge, in second-feet.
1860	602.8	91 400	582.63	227 800
1861	602.9	92 200	582.59	226 800
1862	602.6	86 500	582.56	223 200
1863	602.2	79 900	582.11	213 800
1864	601.9	73 600	581.55	203 400
1835	602.3	81 600	581.29	196 900
1866	602.3	81 200	580.95	187 500
1867	602.5	85 800	581.41	198 400
1868	602.3	82 000	580.91	187 900
1869	602.6	88 400	581.03	190 500
1870	602.4	82 300	581.92	209 500
1871	602.1	74 600	581.94	214 100
1872	602.1	73 800	580.88	193 000
1873	602.5	82 500	581.35	198 800
1874	602.6	82 400	581.80	206 600
1875	602.6	85 200	581.52	205 700
1876	603.0	91 200	582.61	223 300
1877	602.4	82 800	582.40	218 200
1878	601.9	73 600	582.07	211 500
1879	601.4	60 600	581.17	191 500
1880	601.7	69 600	581.28	198 500
1881	602.2	78 300	581.79	205 000
1882	602.2	77 500	582.20	209 000
1883	602.1	73 300	582.43	215 800
1884	601.9	69 900	582.59	217 600
1885	602.2	75 300	582.79	226 800
1886	601.8	70 700	583.01	231 600
1887	601.9	70 200	582.37	217 200
1888	602.2	66 300	581.66	204 000
1889	602.2	67 500	581.21	192 500
1890	602.0	65 200	581.08	185 600
1891	601.7	58 300	580.48	176 400
1892	601.5	57 400	580.38	171 000
1893	601.8	63 000	580.62	177 700
1894	602.6	72 500	580.77	179 700
1895	602.6	76 800	579.78	164 400
1896	602.6	73 800	579.50	155 500
1897	602.6	78 900	580.12	171 100
1898	602.2	72 200	580.30	171 200
1899	602.7	85 700	580.30	174 000
1900	602.7	87 200	580.29	172 300
1901	602.7	87 200	580.55	180 100
1902	602.5	85 000	580.20	167 300
1903	602.7	89 000	580.34	172 500
1904	602.7	87 000	580.78	182 500
1905	602.7	88 000	580.91	185 000
Means	602.31	77 900	581.36	196 400

* The lake elevations in this table are in substantial conformity with the diagram issued by the U. S. Lake Survey. The discharges of St. Mary's and St. Clair Rivers are, for the most part, from the official reports already mentioned.

TABLE 3.—Lake Superior.

Elevation and Outflow Through St. Mary's River.

	1860-1888.		1889-1905.		1860-1905.	
	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.
Lowest monthly mean	600.75 Mar., '79 Mar., '80	49 500 Mar., '79	601.00 Feb., '93	44 700 Mar., '92	600.75	44 700
Highest monthly mean	604.15 Sep., '69	116 600 Sep., '69	603.50 Sep., '99 Oct., 1900	100 400 Sep., '99	604.15	116 600
Range of monthly means	3.40	67 100	2.50	55 700	3.40	71 900
Lowest annual mean	601.35 1879	60 600 1879	601.55 1892	57 400 1892	601.35	57 400
Highest annual mean	603.00 1876	92 200 1861	602.75 1901	87 200 1901	603.00	92 200
Range of annual means	1.65	31 600	1.18	29 800	1.65	34 800
Minimum seasonal range of monthly means, low to succeeding high	0.61 1870	....	0.68 1891	.....	0.61	.....
Maximum seasonal range of monthly means, low to succeeding high	2.67 1869	....	1.84 1899	.....	2.67	.....
Ordinary seasonal range of monthly means	1.28	.....	1.24	.....	1.26	.....
Mean, whole period	602.26	78 900	602.38	76 100	602.31	77 900

TABLE 4.—Lake Superior.

Elevation and Outflow Through St. Mary's River, and Water-Yield of Drainage Basin for Periods of About Five Years.

Period.	Mean elevation, in feet.	Mean discharge of St. Mary's River, in second-feet.	Mean water-yield of drainage basin, in second-feet.
1860-65	602.45	84 200	81 400
1866-70	602.42	83 900	81 400
1871-75	602.38	80 700	87 100
1876-80	602.08	75 600	72 300
1881-85	602.12	74 800	74 000
1886-90	602.02	68 000	68 000
1891-95	602.04	65 600	69 300
1896-1900	602.56	79 600	82 100
1901-05	602.66	87 200	86 900
Means	602.33	77 900	78 100

All elevations herein noted are based on the precise levels of 1903.

The area of the drainage basin of Lakes Huron and Michigan is given as 137 800 sq. miles. The water surface of the two lakes has an area of 45 500 sq. miles.

TABLE 5.—Lakes Huron and Michigan.

Elevation and Outflow Through St. Clair River.

	1860–1883.		1889–1905.		1860–1905.	
	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.	Elevation, in feet.	Outflow, in second-feet.
Lowest monthly mean	580.05 Mar., '69	135 200 Feb., '72	579.00 Dec., '95	110 600 Feb. '96	579.00	110 600
Highest monthly mean	583.60 Jun., '86	272 400 Jun., '86	581.80 July, '89	222 000 Aug. '89	583.60	272 400
Range of monthly means	3.60	138 000	2.80	129 500	4.60	161 800
Lowest annual mean	580.88 1872	187 900 1868	579.50 1896	155 500 1896	579.50	155 500
Highest annual mean	583.01 1886	231 600 1886	581.21 1889	192 500 1888	582.95	231 600
Range of annual means	2.13	43 700	1.71	37 000	3.45	76 100
Minimum seasonal range of monthly means, low to succeeding high	0.34 1879	....	0.35 1895	.....	0.34	.....
Maximum seasonal range of monthly means, low to succeeding high	2.10 1876	....	1.55 1899	.....	2.10	.....
Ordinary seasonal range of monthly means	1.06	.....	1.09	.....	1.08	.....
Mean, whole period	581.91	208 800	580.52	174 100	581.36	196 400

In giving the seasonal range of monthly means in Table 5, the rise from a low to the following high stage alone was taken into account. In the season 1871-72, there was a drop in the water surface of 2.58 ft.

The water-yield of the lake basins in the foregoing tables was determined as follows: In the case of Lake Superior, the annual water-yield is the outflow from the lake, that is, the discharge of St. Mary's River increased by the storage increase, or decreased by the storage decrease, in Lake Superior during the year. The annual water-yield of the entire drainage basin of Lakes Huron and Michigan

(including Lake Superior) is the flow of St. Clair River plus the discharge through the Chicago Drainage Canal, increased by the annual storage increase in these two lakes, or decreased by the storage decrease. The water-yield of the restricted drainage basin of Lakes Huron and Michigan, that is of the drainage basin exclusive of Lake Superior, is found by subtracting the mean annual discharge of St. Mary's River from that of St. Clair River and adding to the remainder the annual storage increase in Lakes Huron and Michigan or subtracting therefrom the annual decrease of storage, as the case may be, and also adding the amount of water diverted into the Chicago Drainage Canal (which was opened in January, 1900, and takes about 4 167 sec-ft. of water from Lake Michigan).

TABLE 6.—Lakes Huron and Michigan.

Elevation and Outflow Through St. Clair River and Water-Yield of Drainage Basin for Periods of About Five Years.

Period.	Mean elevation, in feet.	Mean discharge of St. Clair River, in second-feet.	Mean water-yield* of drainage basin (excl. Lake Superior), in second-feet.	Mean water-yield* of drainage basin (incl. Lake Superior), in second-feet.
1860–65	582.14	215 300	117 000	201 800
1866–70	581.25	194 800	118 100	202 000
1870–75	581.49	203 600	124 500	205 200
1876–80	581.72	208 600	129 000	204 600
1881–85	582.37	214 800	153 300	228 100
1886–90	581.87	206 200	121 700	189 700
1891–95	580.39	173 400	95 800	161 400
1896–1900	580.12	168 800	99 700	179 300
1901–05	580.56	178 300	96 500	183 700
Means	581.36	196 400	117 400	195 200

*The term, water-yield, as here used, means the total delivery of water into the two lakes in excess of evaporation from lake surface.

The normal outflow from Lake Superior through St. Mary's River, as determined from the records covering the 46 years, 1860 to 1905, is 77 900 sec-ft. The mean flow of the river during the period, 1860 to 1888, was 78 900 sec-ft., and, from 1889 to 1905, it was 76 100 sec-ft. The monthly mean flow of St. Mary's River ranges from about 45 000 to about 117 000 sec-ft.

If this lake were converted into a storage reservoir by the construction of works for the regulation of the flow of St. Mary's River,

it would be possible to hold the lake at or near a high stage until the stored water is needed to supply a deficiency in Lakes Huron and Michigan. The degree of benefit that can thus be secured will depend obviously upon the amount of water that can be stored, that is, upon the permissible range from low to high stage of Lake Superior and upon the capacity of the outlet channels. The records for the 46 years, 1860-1905, indicate a range of mean monthly lake stages of 3.40 ft., and a range of the annual means of 1.65 ft. The range of monthly means from low to high in single seasons is normally 1.26 ft., but this amount is frequently exceeded, and there is one season noted, 1869, in which it reached 2.67 ft. This was an unusual fluctuation, and strikingly illustrates the fact that there may be a material departure at certain times each year from the mean annual stage. The probable and possible departure should be carefully studied when works for the complete control of the outflow from the lake are planned.

A description of the works for the partial control of lake stages, made necessary by the construction of the Lake Superior Power Canal, which have been in service since 1902, will be found in *Transactions*.*

By computation in the usual way, with recourse to the mass-curve of the annual water-yield of the Lake Superior basin, it can be shown that under complete regulation, with a range of 1.5 ft. between the extremes of mean annual lake elevations, there would be material improvement over natural outflow conditions. This amount of storage is equivalent to a flow of 42 600 sec-ft. for 1 year. In a succession of seasons such as those from 1860 to 1888, it would keep the outflow at a minimum annual mean of about 69 200 sec-ft. The outflow of 60 600 sec-ft. in 1879 could have been increased by 8 600 sec-ft. From 1860 to 1888, there would have been no time when the mean annual delivery of water from Lake Superior into Lakes Huron and Michigan would have been less than the above indicated minimum of 69 200 sec-ft., unless by intent to conserve water for a subsequent year.

An examination of the mass-curve of water-yield for the entire period, 1860 to 1905, shows the real critical period to have been from 1888 to 1893, in the last three years of which the discharge of St. Mary's River fell to 58 300, 57 400 and 63 000 sec-ft., respectively. A controlled storage capacity of 1.5 ft. in depth over Lake Superior

* "The Compensating Works of the Lake Superior Power Company," by G. F. Stickney, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIV, p. 346.

would have made it possible to increase these amounts to a constant flow of 65 700 sec-ft.

On the assumption that the two critical periods—the one including the years 1878 and 1879 in the first 20 years of the discharge record, and the last-mentioned period—indicate what may be expected in the future, it may be broadly stated that an available storage of 1.5 ft. in depth over Lake Superior would enable the lake outflow to be maintained above a minimum annual mean approximating 65 700 sec-ft. Should it be found practicable to make the effective storage in Lake Superior 2.5 ft. between the extremes of annual mean stages, then the regulation of the discharge from the lake will make it possible to keep the minimum mean annual outflow higher than if storage be restricted to only 1.5 ft. It is found in this case, if past records be again examined, that the critical period in the last 48 years would have extended from 1876 to 1893. In this period of 16 years the regulation of outflow and the addition of stored water would have raised the minimum mean annual discharge from 57 400 to about 71 400 sec-ft.

It is self-evident that under intelligent management the lake outflow would not be kept uniform throughout any series of years nor even throughout any single year. The aim would be to deliver the stored water into Lakes Huron and Michigan in the year and at the season of the year when it would do the most good.

In view of the fact that the natural channel of St. Mary's River will always be supplemented by power canals of large capacity and by the navigation canals on both sides of the river, it seems reasonable to anticipate that the ultimate total capacity of the lake outlets may reach 150 000 cu. ft. per sec. at a mean lake stage. Outflow at this rate is equivalent in one month to a layer of water 0.44 ft. deep on the surface of Lake Superior. In such a year as 1869, in which the lake rose 2.7 ft. in the 6 months from March to September, the lake elevation under complete regulation and this assumed capacity of outlet channels could be held down a little more than 1 ft. below the elevation then reached.

On the other hand, a delivery of water in the amount named from Lake Superior into Lakes Huron and Michigan is equivalent to a layer of water, over the whole surface of these lakes, of 0.30 ft. for each month of such flow. When it is recalled that there has been a year in which St. Mary's River discharged only 57 400 sec-ft., and that

there has been a mean monthly discharge as low as 45 000 sec-ft., it will be readily understood that an assured mean annual delivery in excess of 65 700 sec-ft., and a possible delivery of about 150 000 sec-ft. for a part of the year, will have an appreciable effect on the water stage of Lakes Huron and Michigan. Such regulation will not, as a matter of course, change the mean reduction of water level that would result from the continuous withdrawal, during a long period of time, of a given quantity of water from these lakes; but it will, under intelligent management, result in distributing the amount of depression of the water surface to the several months of the year, and to successive years, so that the lowering of the lakes due to water withdrawal will be least in the months when the lakes are lowest. In other words, by regulating the inflow from Lake Superior, the effect of the withdrawal can be, at least partially, offset at low lake stages. This effect can thus be artificially made maximum when the lakes are high, when there is no injury therefrom to navigation interests, and can be kept at a minimum when lowering would be detrimental to navigation. The effect of controlled water storage in Lake Superior, whether it be restricted to the 1.5 ft. or the 2.5 ft. limits, will be unquestionably of measurable benefit.

The desirability of utilizing Lake Superior as a storage reservoir, together with the utilization of the controlled flow of St. Mary's River to offset the effect of water diversion at Chicago for sanitary purposes, appears to have been pointed out first by Rudolph Hering, M. Am. Soc. C. E., in his report of October 15th, 1907, on the disposal of Calumet sewage.

While it is apparent that in such a year as 1892 the control of storage in Lake Superior would have made it possible to increase the mean annual flow of St. Mary's River from 57 400 to about 65 700 sec-ft., an increase of 8 300 sec-ft., or enough to raise the level of Lakes Huron and Michigan about 2 in. (2.4 in. less the effect of increased flow of St. Clair River due to greater lake elevation), it must be remembered that the effect of this storage upon the mean stages of Lakes Huron and Michigan for a series of years would be comparatively slight. This effect, notable though it may be for a single year or for several years in which stored water increases the flow of St. Mary's River, is offset in a long series of years by the fact that, in the years of more than normal water production, the water delivery into

the lower lakes must be cut down below natural flow, otherwise there would be no water for the improvement of conditions in those years in which the natural outflow from Lake Superior is small. The permanent effect of a controlled outflow from Lake Superior upon the stages of the lower lakes, averaged for a long series of years, would be due mainly to the slight modification of the outflow from these lakes resulting from the modified lake elevation. This effect would be slight and may prove difficult to trace.

Whether an arrangement for storage in Lake Superior, with a range of annual mean elevations as great as 2.5 ft. to secure maximum benefit, can be effected, is not known at this time. It seems possible that this range might be attainable in view of the fact that the extreme range of monthly means (1860-1905) has been 3.4 ft., and that, if the unusually high stage of 1838 be taken into account, this range has been about 4.5 ft. A computation of the effect of the larger storage upon the flow of St. Mary's River has been made, as above set forth, to emphasize the point that controlled storage in Lake Superior can be made beneficial to lake navigation, and, therefore, would be a partial offset to any water diversion from Lakes Huron and Michigan.

Based on the water elevations for the lakes, and the amount of flow in the St. Mary's and the St. Clair Rivers, as noted in the reports of the U. S. Army Engineers, it is found that a continuous withdrawal of 10 000 sec-ft. would lower Lakes Huron and Michigan about as follows:

For a mean annual elevation of 580 ft., the lowering would be 0.49 ft.

For a mean annual elevation of 581 ft., the lowering would be 0.47 ft.

For a mean annual elevation of 582 ft., the lowering would be 0.45 ft.

For a mean annual elevation of 583 ft., the lowering would be 0.43 ft.

A continuous withdrawal of this amount of water during the period 1860 to 1905 would have reduced the water elevations by a mean amount of about 0.46 ft.

As already shown, the offset to this depression, due to storage in Lake Superior during a year, corresponding to a year of minimum

flow of St. Mary's River, such as 1892, would be about 0.17 ft. (storage taken at 1.5 ft.).

The stages of Lakes Huron and Michigan have been lower during the years since 1888 than during the years covered by records preceding that date. Not only has the mean for the whole series of years been lower by about 1 ft., but an extreme low mean annual stage was reached in 1896, which was about 1.35 ft. lower than any recorded low water (annual mean) during the earlier period 1860 to 1888. The year 1888 is a convenient line of division in making this comparison, because the lake stage in that year was about normal, and has since remained less than normal.

The mean lake elevation from 1860 to the close of 1888 was 581.89 ft. The mean lake elevation, 1889 to 1905, inclusive, was 580.45 ft. The lakes were, therefore, about 1.44 ft. lower in the later 17-year period than in the earlier 29-year period. The mean lake elevation determined for the 46 years, 1860-1905, is 581.36 ft. The mean stage of the lakes during the 17 years, 1889-1905, was, therefore, lower than the mean for the entire period by 0.91 ft.

The long-continued depressed stage of Lakes Huron and Michigan has been attributed by the Board of Engineers on Deep Waterways to the enlarged section of St. Clair River and the improvement of the outfall from Lake Huron into the river. On this subject, the Board says:*

"There is now a channel over 40 feet deep from the lake into the river, the increased outflow through which has lowered the general level of Lakes Huron and Michigan about 1 foot."

The Board of Engineers also says:†

"The mean level of Lake Huron is apparently about 1 foot lower than it was fifteen years ago, which change has resulted from the enlargement and deepening of channels for waterway improvements and from the natural erosion of the bed of the river at the outlet of the lake."

The general depth of the foot of Lake Huron, 1½ miles above the head of St. Clair River, is stated by the Board of Engineers to have been originally from 21 to 27 ft., with numerous shoals 16 to 18 ft. deep. A channel, 2 400 ft. wide and 21 ft. deep, at a mean stage, has

* "Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tide Waters," 1900, p. 37.

† "Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tide Waters," 1900, p. 83

been cut through these shoals. Concerning the deepening of water in the head of St. Clair River by the scouring action of the water, the Board of Engineers states that, in 1867, surveys showed the depth of water on the bar over which the lake discharges into the river to have been 27 ft., and the central depth of water in the gorge at the head of the river to have been 48 ft. The surveys of 1898 and 1899, according to the Board's report, showed that a channel had been scoured through the bar to a depth of 75 ft., and that the water depth in the gorge at its narrowest point was from 48 to 66 ft.

The 15 years, 1885 to 1899, inclusive, to which the Board of Engineers referred in making its comparison of lake elevation in recent years with former elevations, show a mean water-surface elevation which was 0.95 ft. lower than that of the 25 years, 1860 to 1884, inclusive.

The possibility that altered conditions at and near the head of the St. Clair River had the effect of modifying, to some extent, the stage of Lakes Huron and Michigan must be admitted, but changes of outlet capacity have probably been only minor factors in producing the low lake stages, if indeed they have been of any effect. This will appear from the following consideration:

It must be apparent that the lowering resulted either from the cause to which it is attributed by the Board of Engineers, or there must have been less water presented by Lake Huron for delivery through the St. Clair River. Both causes may have contributed to the result. If the depressed water surface can be accounted for in part by a decreased water production of the drainage basin which is tributary to St. Clair River, then only the remainder, if there be any, will be ascribed to the increased outflow capacity of the head of St. Clair River.

That there has been a deficient water production in the drainage basin treated as a unit of the three lakes, Superior, Huron, and Michigan, during a long period subsequent to 1888, is primarily indicated by a decrease of the St. Clair River discharge. This decrease is quite as noticeable as the depressed lake elevation, as will appear by inspection of the diagrams on Plate CXV. The flow of the St. Clair River, however, does not by itself represent the water production of the lake drainage basin from year to year, because some of the water remains stored in the lakes. It will be instructive, therefore, to

determine the annual water quantities in excess of evaporation received by the two lakes, Huron and Michigan, from all sources, and to compare these quantities with each other for the same time periods for which comparisons of lake elevations have been made.

The net quantity of water thus annually received by the two lakes is found by adding to the annual outflow through the St. Clair River the storage accretion (plus or minus as the case may be) of the two lakes, and adding also the diversion through the Chicago Drainage Canal, which has been about 4 200 sec-ft. since January, 1900.

The result of this computation, based on the figures contained in the Wheeler and other U. S. Engineer reports, shows conclusively that there has been a very decided falling off in the water-yield of the Huron-Michigan drainage basin in recent years, and that the drop in the water-surface elevation of the lakes is coincident with this decrease of yield. The water-yield of the basin (run-off, and rain on the lakes, less evaporation), as computed, noted for periods of about five years and expressed in second-feet continuous flow, has already been noted in Table 6 and is shown on Plate CXV.

For the 17 years, 1889 to 1905, the entire drainage basin of Lakes Huron and Michigan, including Lake Superior, contributed to the lakes a mean flow of 175 000 sec-ft. of water, whereas the normal water yield of the basin is about 195 200 sec-ft. The deficiency of 20 200 sec-ft. is enough, at the normal stage of the lakes, to account for about 0.94 ft. of deficient water elevation. This being the case, the lakes would have been, at a mean elevation, nearly 1 ft. lower than normal subsequent to 1888, even though outlet conditions from Lake Huron had not changed.

It has already been shown that the mean elevation of the two lakes for the period 1860-1905 was 581.36 ft. and for the period 1889-1905, 580.45 ft. Had there been no withdrawal of water from the lakes at Chicago, these elevations would have been 581.38 and 580.50, respectively. The lakes would have been 0.88 ft. lower, in the 17-year period following 1888, than their normal elevation. This depression of 0.88 ft. is the combined effect of less than normal rainfall (in part perhaps more than normal evaporation) and of changes at the head of the St. Clair River. As above set forth, however, the effect on lake elevation that might be reasonably attributed to the first cause alone, viz., climatic conditions, is about 0.94 ft., slightly in excess of

the actual depression, leaving no drawing-down effect to be ascribed to the changes at the head of St. Clair River.

The conclusion that the channel changes at the head of the St. Clair River have not materially affected the stage of the lakes, if at all, is, as already stated, based on the lake elevation and discharge figures published by the United States Army Engineers. It is noteworthy that these figures show that the material reduction, after 1888, in the water productiveness of the drainage basin of Lakes Huron and Michigan, was confined almost entirely to the area directly tributary to these two lakes. There was no pronounced falling off noted in the discharge reaching these lakes through St. Mary's River from Lake Superior. However, as the published stream measurements and river stages for the same period of 17 years show a decrease in the discharge of St. Lawrence River, the outlet of Lake Ontario, amounting (after correction for depletion of lake storage and the diversion at Chicago) to about 18 000 sec-ft.—nearly as much as the decreased water production of the area tributary to the St. Clair River, 20 200 sec-ft.—there would appear little room for doubting the substantial accuracy of the published discharge tables of the St. Clair River, and, therefore, the conclusion, as stated, relating to the main cause of the low lake stages, seems to be based on reliable premises.

That this conclusion, which attributes the low lake stages of Lakes Huron and Michigan to a long period of deficient water production in the Huron-Michigan basin, and in a very small degree, if at all, to changes at the head of the outlet channel, is probably correct, is borne out by a statement of E. E. Haskell, M. Am. Soc. C. E., in his report of July 16th, 1900,* to the effect that what had a year previously been reported as a clear case of enlargement by scour of the head of St. Clair River was based on a comparison of a preliminary survey of 1898, with a chart based on a survey in 1867, and showed an 18-ft. cut over a portion of the gorged reach. Mr. Haskell goes on to say that an older chart of 1859 was subsequently found to agree much better with the survey of 1898. He caused old notes to be replatted, and careful comparisons based thereon have led him to the conclusion that the changes have been small. Between 1859 and 1867, the most restricted section may have been enlarged 9 000 sq. ft.

* Report of Chief of Engineers, U. S. A., 1900, Pt. 8, p. 5323.

Between 1867 and 1900 the changes, if any, Mr. Haskell says, are unimportant.

On the assumption that the discharge of St. Clair River has been approximated with a fair degree of accuracy for each year of the entire period, 1860 to 1905, it appears reasonably certain that the low stage of Lakes Huron and Michigan will not persist, that normal weather conditions will restore about 1 ft. of the lost mean lake elevation, and that the only depression below the original normal will be a small amount, if any, due to the St. Clair River changes and a small amount due to the Chicago diversion. Until the Chicago diversion is increased above the present amount of about 4 200 sec-ft., a mean lake stage at about 581.20 ft. is to be expected. This, it may be stated, is 1.70 ft. higher than the lowest mean annual lake elevation (579.50 ft.) of the past 48 years, which occurred in 1896.

This conclusion relating to a probable future higher lake level than that of the period subsequent to 1888 is inevitable, because it may be accepted as a certainty that the unusual climatic conditions which, since 1888, have resulted in a deficient output of water from the Huron-Michigan basin will not continue indefinitely. The occurrence of another period, as protracted as the one subsequent to 1888, of small water-yield in the lake basin is highly improbable. It is proper, therefore, to assume that the years in which such low levels as those of 1895 and 1896 will occur will be few and far between.

What has occurred in the past, however, may occur again; therefore means should be sought to keep up the elevation of Lakes Huron and Michigan during any future periods of deficient precipitation in the lake basin.

To some extent the raising of the level of these lakes can be accomplished with controlling works in the head of Niagara River. Such works have been recommended, and designs therefor have been made by the Board of Engineers on Deep Waterways. This Board proposes a submerged weir 2 900 ft. long and a series of sluice-gates, 13 in number, and each 80 ft. wide. These works, it is stated, would raise the level of Lake Erie 3 ft., the level of Lake St. Clair about 2 ft., and the level of Lakes Huron and Michigan about 1 ft. No attempt has been made to check this forecast. It is possible, moreover, to go a step farther and to provide works for the throttling of the waterway, or for the complete control of flow, within fixed limits, of

the St. Clair River. The stage of Lakes Huron and Michigan could then be brought under adequate control, and the lake stage could be maintained at all times as high as required by navigation interests.

That regulating works will sooner or later be constructed, in the rivers draining the several lakes, of such a character that the elevation of the lakes under proper management cannot fall below predetermined minima, is reasonably certain. It follows, therefore, in view of the benefit that will be derived from the use of Lake Superior as a storage basin, and of the control of water surface that will result from works below Lake Huron, that navigation interests will be but temporarily, if at all, affected by a diversion of water at Chicago, or elsewhere within reasonable limits as to amount, and that any deleterious effect will not continue beyond the time when the water elevation of this lake and Lake Michigan will be controlled by works below Lake Huron.

The stages of Lake Ontario and of the St. Lawrence River are questions apart from the one that has here been discussed. Enough has been said, however, to show that here, too, the less than normal stages of recent years are to be ascribed to climatic conditions, supplemented in a very slight degree only by the diversion at Chicago. The ultimate effect of the diversion of water from any of the lakes, offset by the equalizing effect of lake regulation, upon the stage of water in the St. Lawrence River is a study that will have to be made when the regulating works come under consideration.

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PAPERS AND DISCUSSIONS

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FOUNDATIONS FOR THE NEW SINGER BUILDING, NEW YORK CITY.

By T. Kennard Thomson, M. Am. Soc. C. E.

To be Presented November 18th, 1908.

In August, 1906, the Singer Manufacturing Company awarded a contract to The Foundation Company of New York City, for sinking the pneumatic foundations of the addition to the old building on the northwest corner of Broadway and Liberty Street, now known as the Singer Tower.

This contract was for a lump sum, on the basis of the foundations being carried down to 70 ft. below the curb, or approximately to the average depth of the top of the hardpan as shown by the borings; for so much per cubic yard for everything between the depths of 70 and 75 ft.; and for an additional price per cubic yard for everything below 75 ft. The price to be deducted in case the depth of 70 ft. was not reached was about one-half of the price to be added. The progress reports of each caisson, given in Table 2, Plates CXXIV and CXXV, will

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

show the justice of this proportion. The contract stipulated that the work should be completed in 110 days on the same basis of depths.

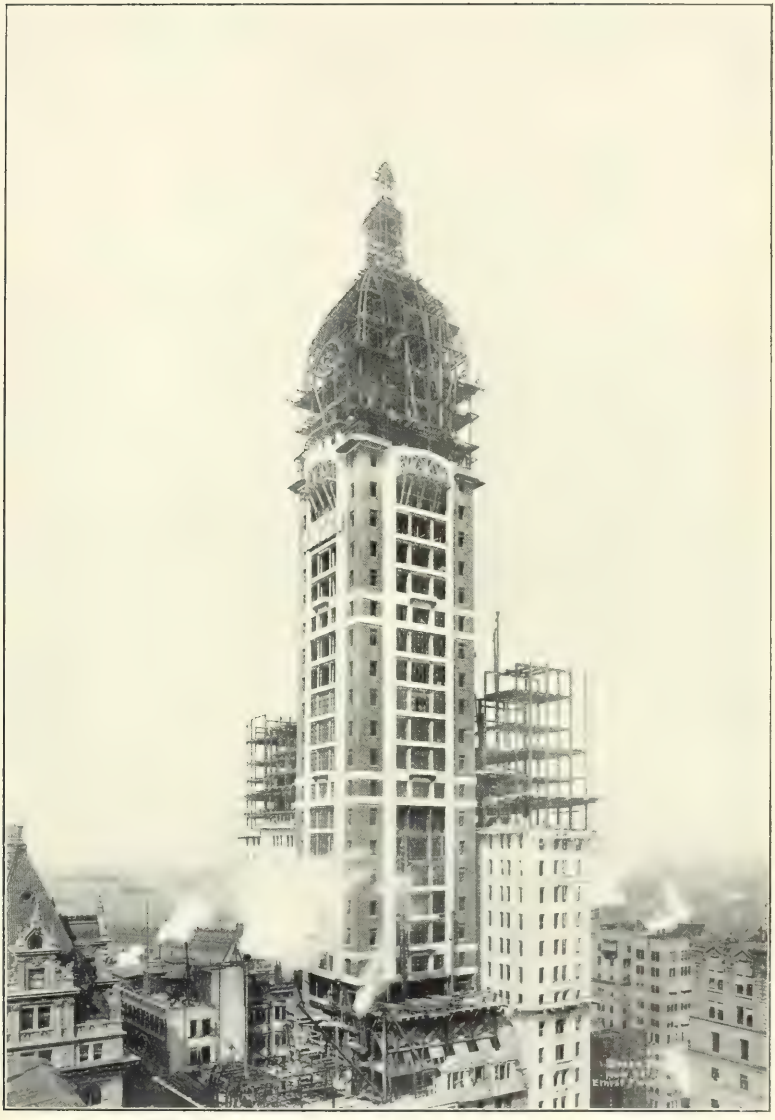
This is not only the fairest method of letting caisson work, but it is the only method which should be adopted; for if the contract is made for a lump sum to bed-rock, either the contractor must bid excessively high to allow for uncertainties, or run a good chance of losing his anticipated profits and much more besides. This is especially true where the borings are what are known as wash borings, which scarcely ever reach rock, but stop at the hardpan. This statement applies at least to wash borings made for sky-scrapers, although it does not apply to some which have been made for some of the railroads, where the borings have reached bed-rock by the use of an occasional small charge of dynamite.

Diamond-drill borings, of course, will find rock accurately, but only for the exact spot of the drill hole; for bed-rock here, known as New York gneiss, is very irregular, in fact it was found to be 6 ft. higher in one caisson than in another, although the two caissons were only 12 in. apart, each landing on a comparatively flat rock. In another case we landed one end of a caisson, 14 ft. long, on bed-rock and then had to dig 14 ft. deep at the other end to find rock. This is mentioned because the surface of the rock under the Singer Building is exceptionally level, as shown by the cross-sections of the caissons, Fig. 1.

The size of the lot is approximately 75 ft. on Broadway and 115 ft deep on the south side.

The tower is square in cross-section and has 36 columns, all 12 ft. center to center, making it 60 ft. square—measuring from center to center of columns. These tower columns rest on 20 pneumatic caissons, as shown on Figs. 2 and 3, and are carried to bed-rock; Caisson No. 48-49, the second to be sunk, was carried to rock as an exploration caisson. The remaining caissons are not under the tower, and were stopped in good hardpan.

At first it was intended to stop all the caissons as soon as good hardpan was reached, but after several had been sunk, including Caisson No. 30-43, which is one of the tower caissons, it was decided that all those under the tower should be carried to bed-rock, and the question arose as to what could be done about Caisson No. 30-43, which



TOWER OF THE SINGER BUILDING DURING CONSTRUCTION.

31–32
Poor Hard Pan
Good Hard Pan
Poor Hard Pan
Sand
Rock
25–26
Poor Hard Pan
C.E. 72′ 6″
Good Hard Pan
Rock
19–20
Quicksand
C.E. 69′ 1″
Poor Hard Pan
Good Hard Pan
Rock
13–14
Quicksand
C.E. 68′ 1″
Poor Hard Pan
Good Hard Pan
Rock
7–8
Quicksand
El. C.E. 69′ 1″
Poor Hard Pan
Good Hard Pan
Rock
1–2
Quicksand
Poor Hard Pan
C.E. 67′ 6″
Good Hard Pan
Rock
3–9
Quicksand
El. C.E. 72′
Poor Hard Pan
Good Hard Pan
Sand
Rock
70′ Below Curb
80′ Below Curb
90′ Below Curb
15–21
Quicksand
C.E. 72′ 7″
Sand
Sand
Sand
Good Hard Pan
Rock
16–22
Quicksand
C.E.
Good Hard Pan
Rock
4–10
Quicksand
C.E. 71′
Poor Hard Pan
Good Hard Pan
Sand
Rock
5–6
C.E. 72′ 5″
Gravelly Hard Pan
Poor Hard Pan
Good Hard Pan
Loose Rotten Rock
Sand
Rock
11–12
Quicksand
C.E. 70′ 5″
Poor Hard Pan
Good Hard Pan
Sand
Rock
17–18
Quicksand
C.E. 70′ 6″
Poor Hard Pan
Good Hard Pan
Sand
Rock
23
Quicksand
C.E. 70′
Good Hard Pan
Sand
Rock
70′ Below Curb
80′ Below Curb
90′ Below Curb

FIG. 1.

had already been carried 7 ft. into the hardpan and filled with concrete. The contractors volunteered to tunnel under this caisson at their own expense, so that the entire tower would rest on bed-rock, and this was done successfully as follows:

When Caisson No. 29-35 had been sunk some 5 ft. below the bottom of No. 30-43 (Fig. 4), a drift was cut under and to the north end of the latter. This drift was about 4 ft. wide and 5 ft. high, and when it had been completed all the hardpan under the north end of the caisson was removed (Fig. 5), the distance from the bottom of the old concrete to the top of the rock being 15 ft. at this point.

The hole under the north end was filled at once with concrete which was rammed up against the old base (Fig. 6). This, by the way, gave the opportunity to remove, from the original base, a piece of the concrete, put in under air, and this, having been made quite wet in the first place, was found to be very hard and compact. It is well known to caisson men that concrete when made properly with plenty of water sets very quickly in compressed air and becomes very hard.

As soon as the north end of Caisson No. 30-43 had been carried to rock, as before described, the adjoining caisson, No. 29-35, was excavated to rock, and the remaining hardpan under the south end of No. 30-43 was removed and the space concreted, thus completing, without any accident, probably what has been the only attempt ever made to undermine a pneumatic caisson (Fig. 7). Of course, if the original excavation had not been carried some 7 ft. below the cutting edge into good hardpan to which the concrete had firmly united, much more difficulty would have been encountered, if indeed the attempt had been made.

It might be stated that the cutting edge of a caisson is rarely carried much below the top of the hardpan, and it should not be, for, if the excavation is carried below the cutting edge into the hardpan and the space then filled with concrete, the concrete forms an excellent bond with the hardpan, greatly reducing the load on the base, which is usually from 12 to 15 tons per sq. ft. Whereas, if the caisson is carried through the hardpan there would be almost no friction below the top of the hardpan.

In this case the foundations were designed for 15 tons per sq. ft. In designing the foundations, it was decided that, if the wind pressure did not exceed 50% of the dead and live load, it would not be regarded, so the wind strains were not considered.

FIG. 1.—AUGUST 28TH, 1906, PREPARING TO BUILD PLATFORM.

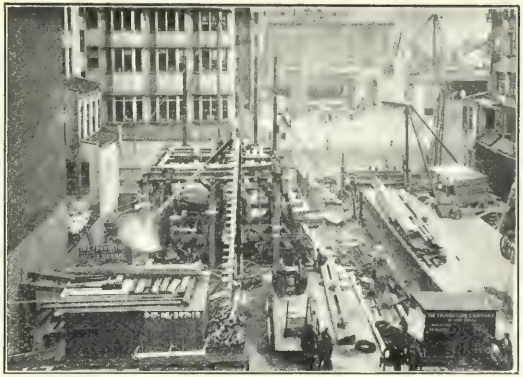


FIG. 2.—SEPTEMBER 17TH, 1906, PLATFORM PARTLY BUILT; ERECTION OF DERRICK COMMENCED.

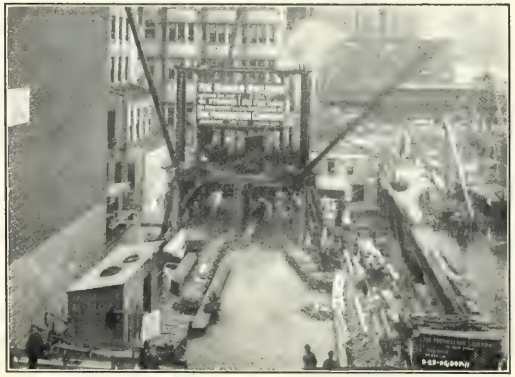


FIG. 3.—SEPTEMBER 29TH, 1906, DERRICK COMPLETED AND IN USE.

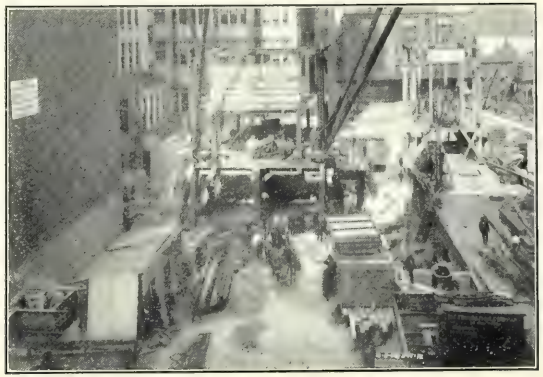
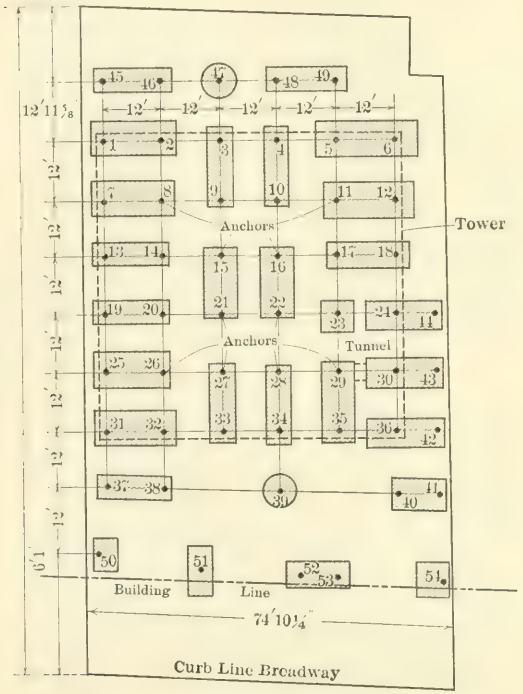
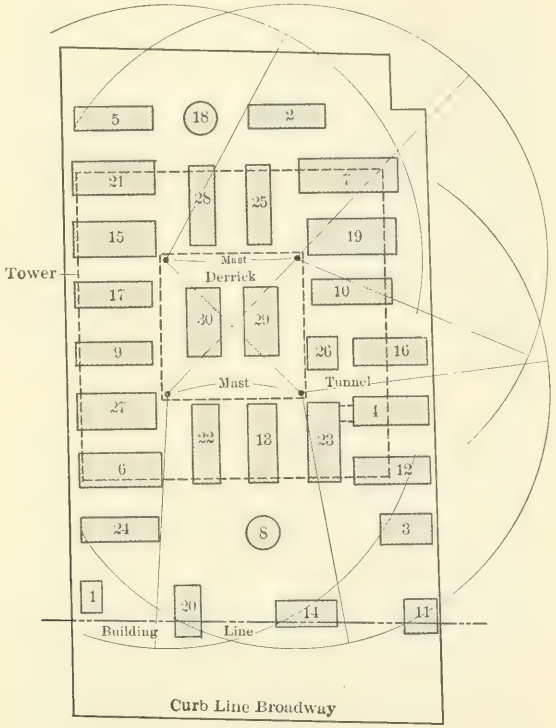


FIG. 4.—OCTOBER 8TH, 1906, CONCRETE BEING PLACED.



Location and Number of Columns, Caissons being designated by Column Numbers.

FIG. 2.



Order in which caissons were sunk.

FIG. 3.

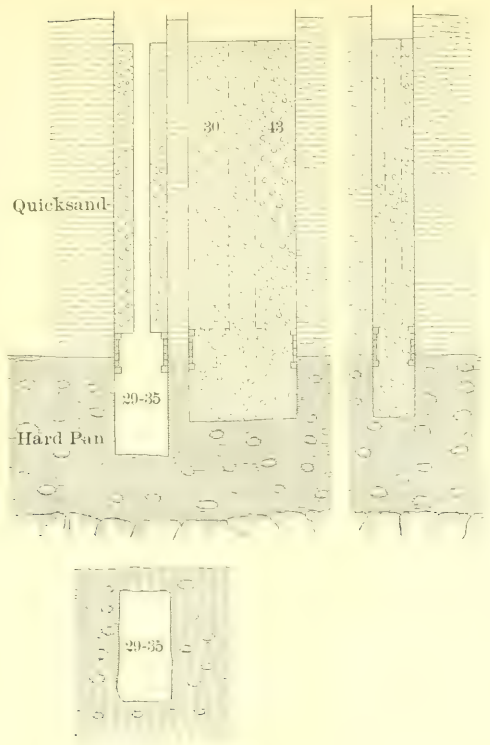

30
43
Quicksand
29-35
Hard Pan
29-35

FIG. 4

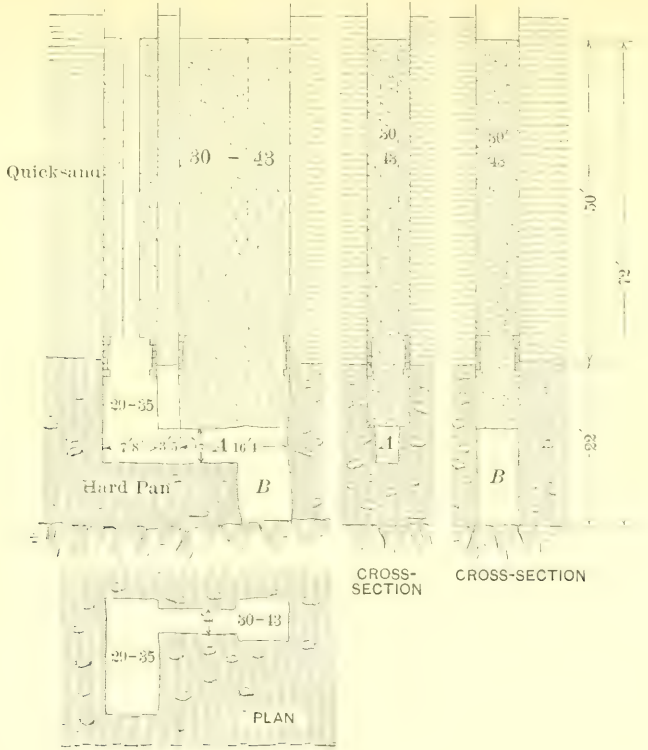

Quicksand
30 - 43
50
45
50'
45
22'
29-35
16'-1"
Hard Pan
B
B
CROSS-SECTION
CROSS-SECTION
30-43
29-35
PLAN

FIG. 5.

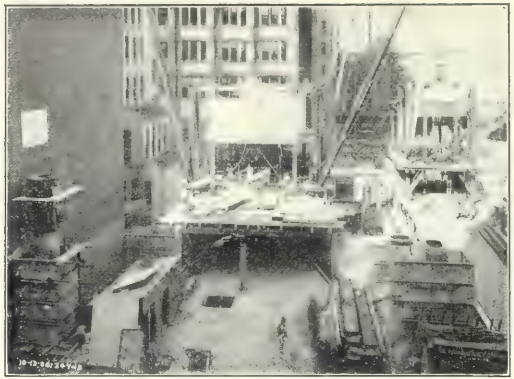


FIG. 1.—OCTOBER 13TH, 1906, TEMPORARY FORMS FOR CONCRETE ON TOP OF CAISSONS.

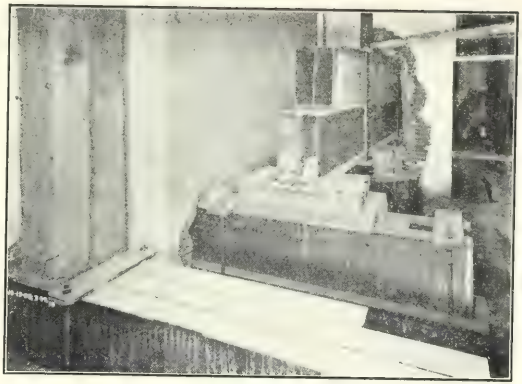


FIG. 2.—NEEDLE BEAMS SUPPORTING OLD BUILDING.

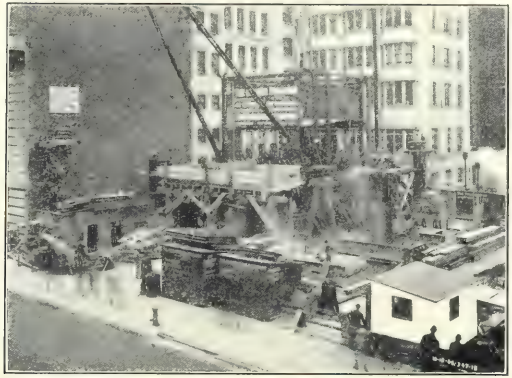


FIG. 3.—OCTOBER 18TH, 1906, SHOWING SEVERAL CAISSONS IN PROGRESS.

FIG. 4.—NOVEMBER 12TH, 1906, SHOWING PROGRESS.

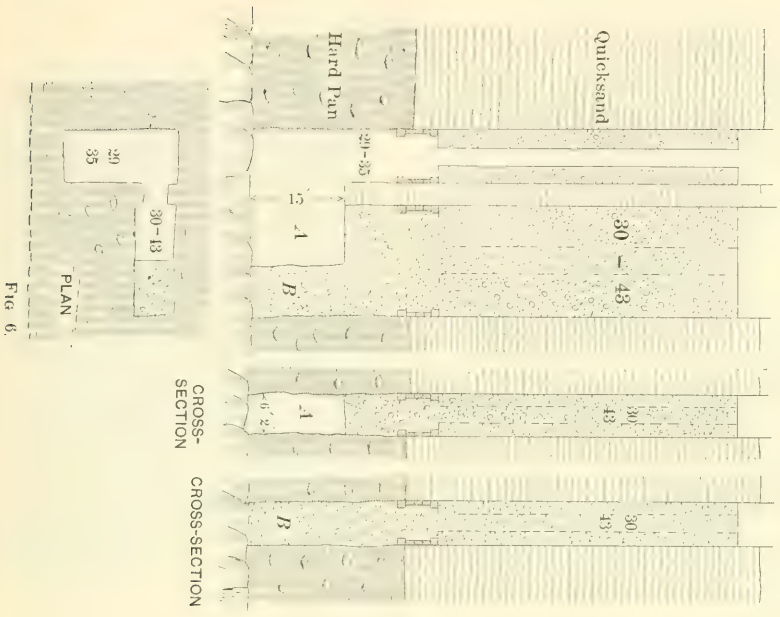

Quicksand
Hard Pan
20-35
15
A
B
30
43
CROSS-SECTION
CROSS-SECTION
PLAN
30-43
20-35

Fig 6.

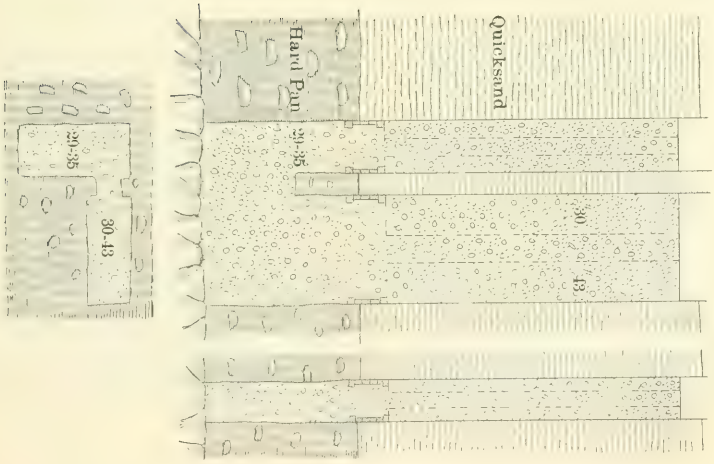

Quicksand
Hard Pan
20-35
30
43
30-43
20-35

Fig 7

The following are some of the principal facts relating to the Singer Building:

Height of tower from bottom of caisson to top of flag-pole	745 ft.	
Height from basement floor to top of flag-pole	666 "	
Height from sidewalk to top of flag-pole	652 " 6 in.	
Height of tower from sidewalk to top of lantern	612 " 3 "	
Height of main building from sidewalk to roof	191 " 8 "	
Greatest depth of caisson below curb	92 "	
Area of each main floor	20 163 sq. ft.	
Area of each floor of tower	3 737 " "	
Total area of floor	411 333 " "	or 9.44 acres.

Plant.—The hoisting plant consisted of a four-boom derrick and two stiff-leg derricks, with five Lidgerwood double-drum engines, 3 ft. 7 in. by 10 in. and two, 8¼ by 10 in., and one Lambert, 7½ by 10 in.; in addition, there were four Rawson and Morrison boom-swinging gears.

A platform was built on the level of the sidewalk about 15 ft. above the excavation. It is customary to excavate the lot to about the water level before commencing the caisson work proper, and the derrick was built so that carts could run under it on the street level. It was about 30 ft. square, with four masts 30 ft. high and 50-ft. booms.

The compressor plant consisted of one Rand straight-line compressor of 14-in. steam cylinder by 18-in. air cylinder by 22-in. stroke, capable of pumping 1 294 cu. ft. of free air per minute, theoretical rated capacity, and one McKiernan 22-in. steam cylinder by 26-in. air cylinder by 24-in. stroke compressor, capable of pumping 1 474 cu. ft. of free air per minute with a speed of 100 rev. per min. Twin air receivers were used, each 41 in. in diameter, coupled, and 15 ft. 9 in. long. There was also a 14-in. air cooler, 14 ft. long.

The concrete was mixed in one Ransome mixer with a capacity of 24 cu. ft., and one Chicago mixer with a capacity of 18 cu. ft., both placed under the street platform.

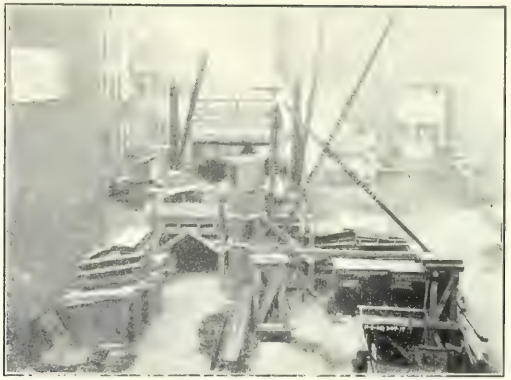


FIG. 1.—NOVEMBER 8TH, 1906, SHOWING PROGRESS.

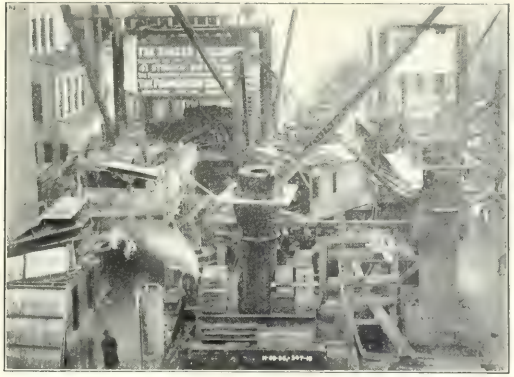


FIG. 2.—NOVEMBER 20TH, 1906, SHOWING PROGRESS.

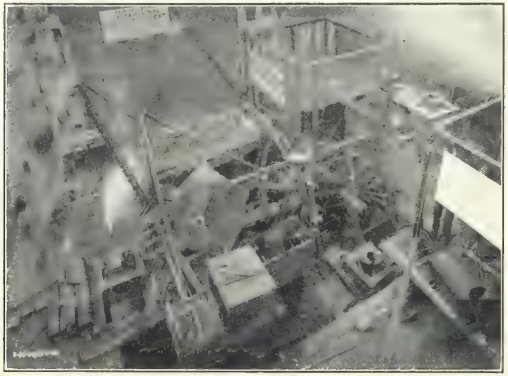


FIG. 3.—NOVEMBER 30TH, 1906, BIRD'S-EYE VIEW.

FIG. 4.—COLLAPSIBLE SHAFTS AND BOTTOM-DUMP BUCKETS.

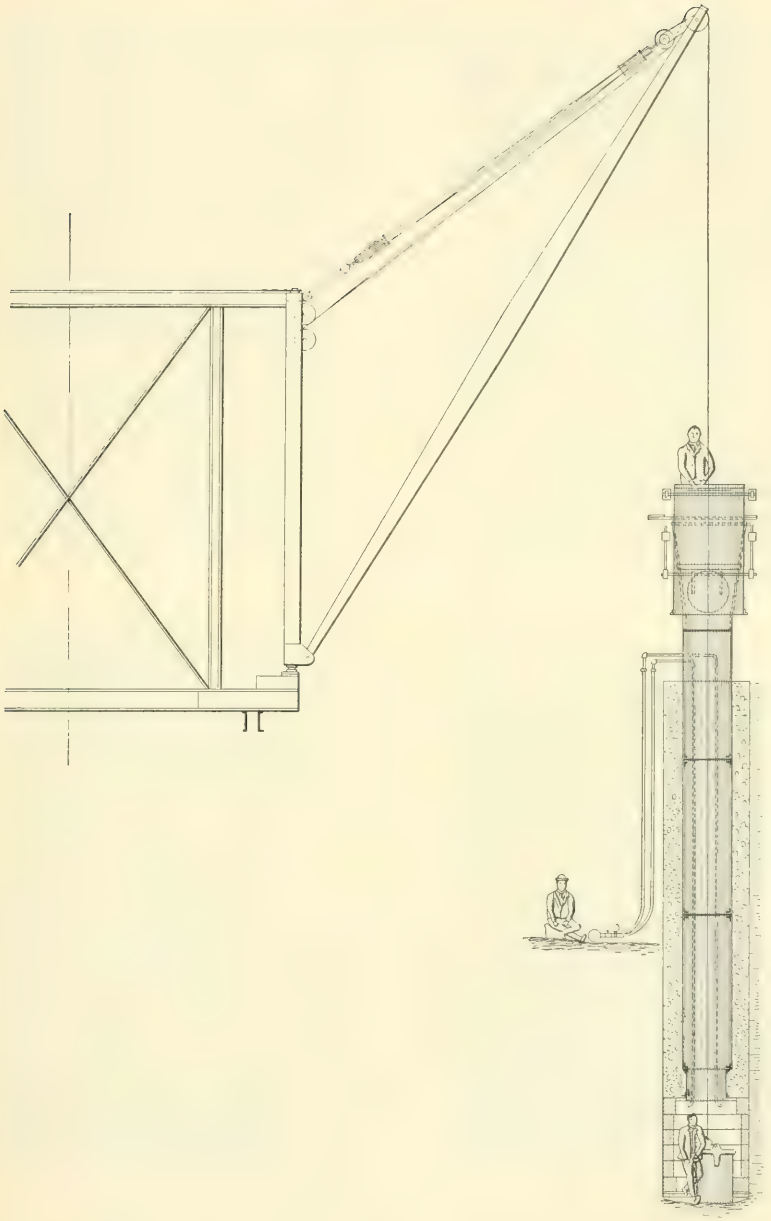


FIG. 8.

Fig. 1, Plate CXVII, shows the ground on August 28th, 1906, with the contractors getting ready to build the platform. It will be noticed that they had already stored some 800 tons of pig iron on the site, ready for use, in addition to 400 tons of cast-iron blocks, each block weighing $1\frac{3}{4}$ tons, or as much as 70 or 80 pieces of pig iron.

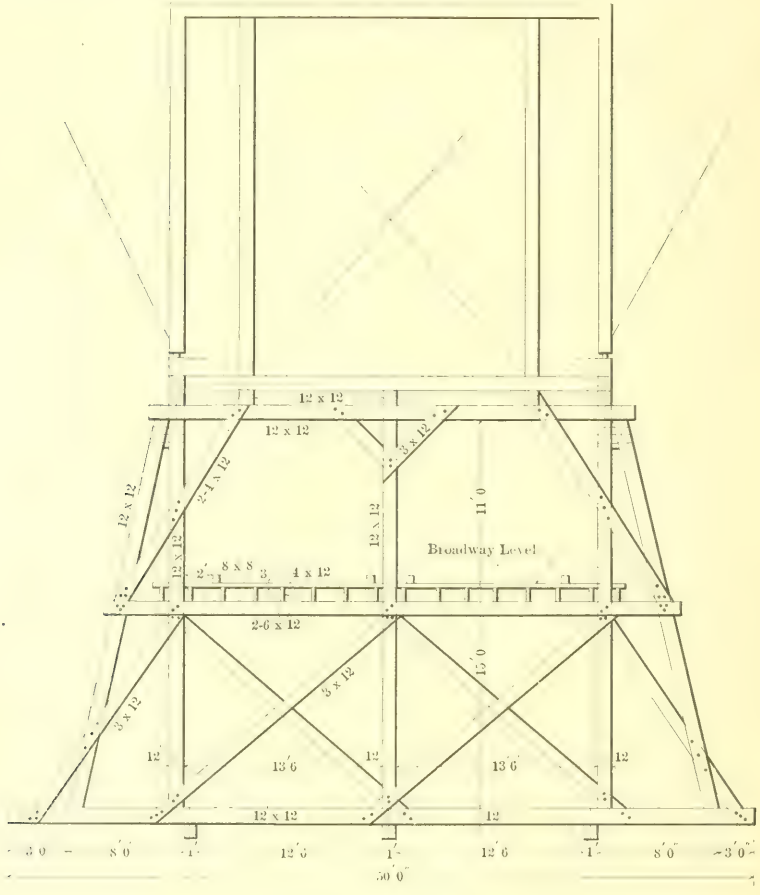


Fig. 9.

By September 17th, the platform had been partially built on the Broadway level and the erection of the four-masted derrick commenced. Fig. 2, Plate CXVII, taken from Broadway, shows a temporary stiff-leg derrick with a gin-pole derrick in the rear used for the erection of the four-boom derrick.

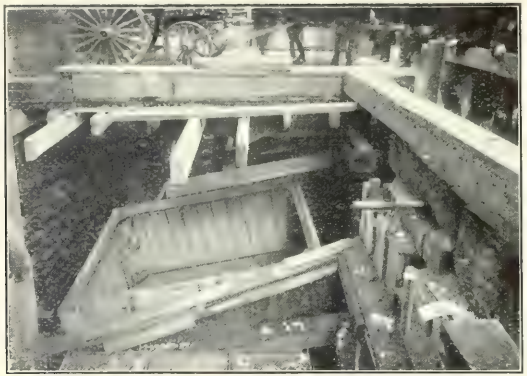


FIG. 1.—COFFER-DAM ON TOP OF CAISSON FORMS.

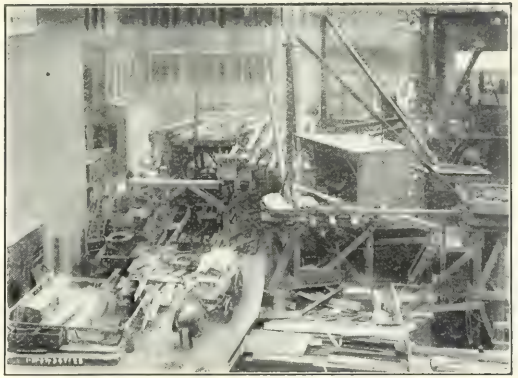


FIG. 2.—JANUARY 21ST, 1907, REMOVING THE FOUR-MASTED DERRICK.

FIG. 3.—JANUARY 23D, 1907, LOOKING TOWARD BROADWAY, SHOWING PROGRESS.

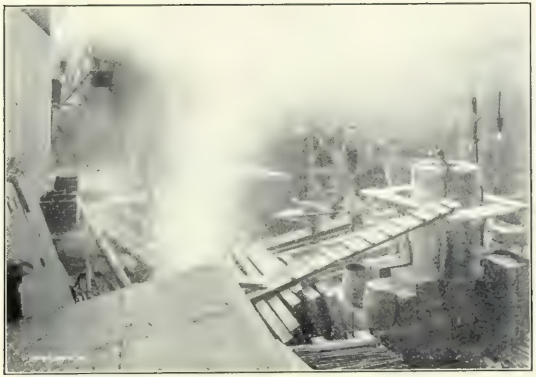


FIG. 4.—JANUARY 23D, 1907, SHOWING PROGRESS.

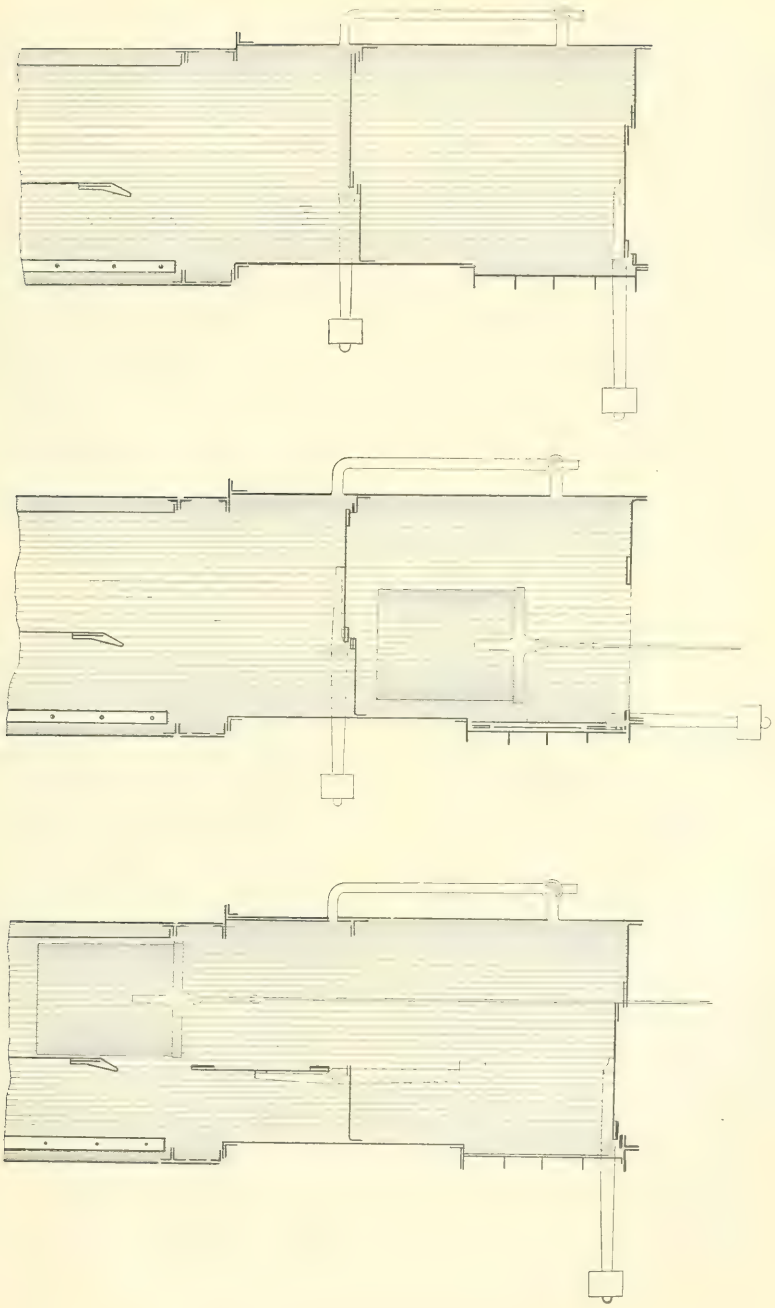


Fig. 10.

Fig. 3, Plate CXVII, taken on September 29th, shows the four-boom derrick completed and in use, the two temporary derricks having been removed.

Fig. 4, Plate CXVII, taken on October 8th, 1906, shows the caisson for Column No. 50, the first to be sunk into position, being concreted around the shaft which is covered up to prevent the concrete from falling into the working chamber. It also shows the caisson under Columns 40-41 to the right, in place, which was the third to be sunk, the second caisson being behind the derrick. It also shows another caisson resting on the I-beams on the platform.

Fig. 1, Plate CXVIII, taken on October 13th, 1906, gives a good idea of the temporary forms of 2-in. plank, tongued and grooved, and planed, used for the concrete on top of the caissons. As will be seen, the steel angles, usually 3 by 3 by $\frac{3}{8}$ in., are not cut to length for each size of caisson, but are allowed to project beyond the corners. It also shows the air-lock on the first caisson sunk—that under Column No. 50—the photograph having been taken two days before air was put on.

The four-masted traveler to the right is one of the traveling derricks on the lot of the City Investing Building, which was under construction at the same time as the Singer Building, the writer being retained on both. Some interesting problems were encountered, one of which was to locate the cutting edges of the caissons on each side of the lot line. Owing to the caissons not being strictly plumb, those of each building encroached on or under the adjoining property. Fortunately for the owners, both sides were trespassers, otherwise a lawsuit might have given much work to the lawyers and engineers.

Most specifications state very positively that a caisson shall not be out of plumb more than a certain amount, varying generally from 1 to 6 in.; but suppose a caisson is down 90 ft. in the ground and is found to be, say, 7 in. or more out of plumb—in fact, the writer has heard of, but has not seen, a caisson which was 5 ft. out of plumb—what can be done about it? Of course, a divergence of an amount like 5 ft. is absolutely inexcusable, but the best care will not prevent an error of a few inches. The only remedy is prevention—for an experienced man can start his caissons right and keep them so, and it is to the owner's advantage to attend to this from the start. If a caisson gets much out of plumb and down to a certain depth, it is

FIG. 1.—OCTOBER 26TH, 1906, SHOWING LACK OF SPACE.

FIG. 2.—CAISSON JUST LIFTED OFF TRUCK.

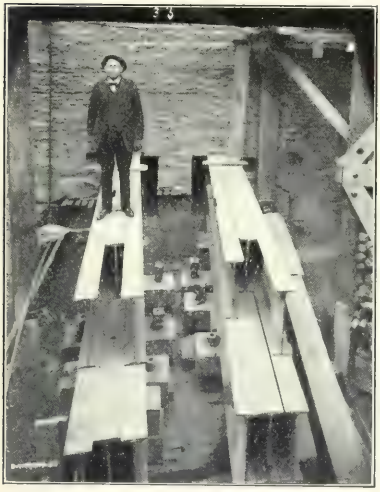


FIG. 3.—NEEDLE BEAMS AND SCREW-JACKS USED IN UNDERPINNING OLD BUILDINGS.

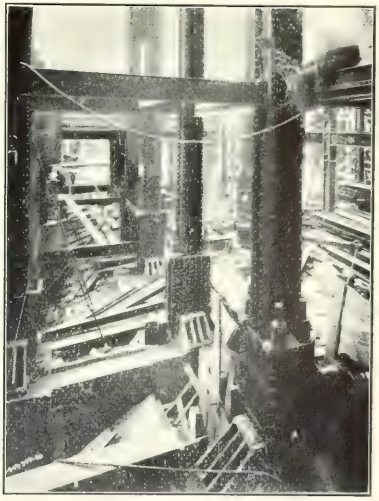


FIG. 4.—APRIL 5TH, 1907, STEELWORK BEING ERECTED.

not only impossible to plumb it again, but it is also impossible to prevent it from getting rapidly worse. The best superintendents rely on the engineers to keep them posted as to how the caisson is going, for the superintendent who tries to do this for himself with a hand level or plumb line will be much astonished to find how far he is out. This is the reverse in building masonry, for a good mason, when started right, can generally be relied on to build a pier absolutely to lines.

Fig. 2, Plate CXVIII, taken on October 13th, 1906, shows the needle-beams used to support the old building. As the underpinning has been described in the technical press, it will not be enlarged upon here.

Fig. 3, Plate CXVIII, taken on October 18th, shows the work in full blast; Caisson No. 50 to the left is taking a rest while the concrete is hardening in the forms. Caisson No. 48-49, the second to be sealed in under air, is shown in the rear, with heavy cast-iron weights piled on top of the concrete. It also shows Caisson No. 30-43 on the day when the compressed air was turned on, and Caisson No. 31-32, on which the lock is being placed. As has been stated, some 1 200 tons of cast-iron weights and pig iron were in use on this job at one time, and it can be readily understood what an expensive item it was. The great advantage of the cast-iron blocks, weighing $1\frac{3}{4}$ tons each, is the saving in time in handling. The only disadvantage is the necessity of using the derrick when it may be wanted for other purposes.

Fig. 1, Plate CXXI, gives a good idea of one of the serious difficulties which caisson men experience in city work, namely, lack of space; in fact, this is shown in nearly all the other plates. Here was a lock resting on the dumping platform, piles of temporary forms on the roadway, etc., and yet the contractors were obliged to be continually hauling plant, etc., back to the yard, to be returned when needed, perhaps in a few days.

Fig. 4, Plate CXVIII, taken on November 12th, 1906, shows where part of the dumping platform has been cut away for Caisson No. 28-34, the thirteenth to be sunk. It also shows 12 by 12-in. timbers piled up under the dump and a section of shafting in the corner, with forms everywhere—on top of the shanties, leaning against the wall, etc.

Fig. 2, Plate CXIX, taken on November 20th, shows a similar

condition, the only space left clear being room for a team to drive from Broadway, under the derrick, where it could be turned around by a good driver.

Fig. 3, Plate CXIX, a bird's-eye view, makes the unavoidable jumble look even worse.

Fig. 2, Plate CXXI, shows a caisson which has just been lifted off the truck and gives an excellent idea of its construction. The bolts shown extend down to the cutting edge, and are 1 in. in diameter and about 3 ft. apart; the temporary 2-in. plank roof, on which the men are standing, was removed about 48 hours after 2 ft. of concrete had been placed upon it. Fig. 2, Plate CXXI, also shows the special provision made for "hooking on" to the caisson for hoisting, which is much neater and better than the old way—still often seen—of wrapping around the caisson a rope or chain which was always likely to slip and injure it.

The cutting edge is a 6 by 4 by $\frac{1}{2}$-in. angle with the 6-in. leg horizontal; on top of the cutting edge is a horizontal course of 8 by 12 in. (12 in. vertical), then four courses of 6 by 12 in., on top of which is a 10 by 12-in. course. The longer caissons, of course, have struts and ties in the air-chamber.

Fig. 4, Plate CXIX, taken on January 7th, 1907, gives Broadway a rather unnaturally deserted appearance, but gives a good end view of the combination collapsible shafts, and under them some bottom-dump buckets.

Fig. 1, Plate CXX, taken on the same day, shows the coffer-dam on top of the forms of a caisson, but the writer can vouch for the fact that no caisson on this work was so much out of plumb as this picture would indicate; which goes to prove that the camera is sometimes quite a liar.

Fig. 2, Plate CXX, taken on January 21st, 1907, shows the four-masted derrick nearly all removed, the 28th caisson having been completed on January 18th, leaving only two to be done, and, as these were to be directly under the derrick, they could not be placed until the derrick and its platforms had been removed, so it was January 23d and 24th, respectively, before these two caissons were located, and the 13th and 19th of February before they were completed. Thus the work was done in 154 days from the time of signing the contract—the 110 days of the contract being understood to cover the time to be

FIG. 1.—JANUARY 29TH, 1907, SPECIAL DERRICK FOR HANDLING THE LAST TWO CAISSONS.

FIG. 2.—FEBRUARY 16TH, 1907, SHOWING CONSTANT CHANGES IN PLATFORMS AS WORK PROGRESSED.

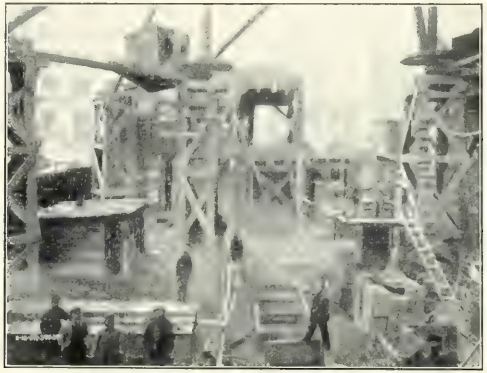


FIG. 3.—FEBRUARY 16TH, 1907, PLACING THE LAST CAISSON.

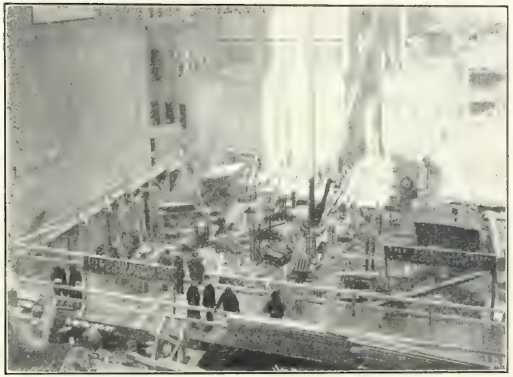


FIG. 4.—MARCH 16TH, 1907, STEELWORK COMMENCED.

taken if the sinking stopped at Elevation 70; but all the caissons went below this depth, many below 90 ft., and the additional 20 ft. of sinking consumed more time than the upper 70 ft.

Fig. 3, Plate CXX, was taken on January 23d, 1907, just before the 29th caisson reached the site. Caisson No. 3-9 (the 28th finished), though completed, still shows the lock and cast-iron weights in place. This view was taken looking toward Broadway, whereas the previous views were taken looking from Broadway.

Fig. 1, Plate CXXII, taken on January 29th, shows an additional derrick, erected on three high trestles, just for handling these last two caissons.

Fig. 3, Plate CXXI, shows the needle-beams and screw-jacks used in underpinning the old building; 40 and 60-ton screw-jacks were used.

Fig. 2, Plate CXXII, of February 16th, 1907, gives a fair idea of the constant changes in the platforms, etc., as the work progressed from day to day.

Fig. 3, Plate CXXII, taken on February 16th, 1907, looking toward Broadway, shows the last caisson three days before the air was taken off. It also shows how rapidly the place was being cleaned up in order to set the bases, finish the concreting, etc.

One month later, that is, on March 16th, 1907, Fig. 4, Plate CXXII, shows that all the derricks of The Foundation Company have been removed, and that a big guy derrick for the iron erectors—Messrs. Milliken Brothers—has been put in place; the anchor-bolts, bases, etc., are everywhere in evidence.

Fig. 1, Plate CXXIII, of March 25th, shows two of these guy derricks in place, and Fig. 2, Plate CXXIII, and Fig. 4, Plate CXXI, of April 5th, show a very different scene, the erection of the steel-work being well under way. These two views show very plainly how the columns were anchored to the caissons, the detail of the anchors being shown on Fig. 11. Mr. O. F. Semsch designed these with sections decreasing from the top down to the bottom, at 60 ft. below the curb, the idea being to save the weight of the anchors by counting on the adhesion of the concrete to the steel bars, using 50 lb. per sq. in. for adhesion, as allowed by the New York Building Department.

Thus, for the bottom 10 ft., Mr. Semsch used one bar varying from 6 by $1\frac{1}{2}$ in. to 6 by $3\frac{3}{8}$ in.; for the next $10\frac{1}{2}$ ft. above, he used two

bars varying from 6 by 1¼ in. to 6 by 2½ in., and coupled to the lower bar by pins 6½ in. in diameter; for the next 9½ ft., or from 40 ft. to 30½ ft. below the curb, he used three bars varying from 6 by 1 in. to 6 by 2⅜ in.; then four bars to 22 ft. below the curb, these running from 6 by 1 in. to 6 by 1¾ in. At this point he had a saddle with a 7¼-in. pin which connected the four flat bars with four round rods, which were 2¾ in. in diameter where the uplift was 270 tons, and 3⅝ in. in diameter where it was 480 tons. These are the round rods projecting above the column bases as shown in the plates. They were ordered 2 ft. longer than the calculated length so as to allow for any variation in the depth of the caisson, etc.

In three of the columns (Nos. 11, 16, and 22), the base was 8½ ft. lower, so the four top flat bars were omitted, the saddle being placed on top of the three flat bars.

The columns anchored were interior columns, ten in all, viz., Nos. 8, 11, 15, 16, 21, 22, 26, 27, 28, and 29.

Table 1, showing loads on foundations, is made up from figures of Messrs. Boller and Hodge.

By the courtesy of The Foundation Company, the writer was allowed access to all records of construction, and, therefore, has been able to give, in Table 2, the complete record of each individual caisson. This table shows the number of caissons on the site and the number under air at any one time. It also shows the difference in time consumed in going through quicksand and hardpan; for instance, in Caisson No. 17-18, the tenth to be sunk, 50½ ft. of sand were penetrated in 62 hours, while it took 91 hours to go through 20 ft. of hardpan. The writer has seen a caisson penetrate only 6 in. in 24 hours, where boulders, etc., were encountered.

It will be noticed that the total number of hours under compressed air is very much greater than the total number of hours of actual excavation and concreting. This was due to the fact that forms were used above the deck instead of coffer-dams, requiring the excavation to stop until the concrete was hard enough to allow the forms to be taken off safely.

Where coffer-dams are used, the sinking can be kept up continuously, even if the concrete is being placed below the ground level, which, of course, is impossible with forms which have to be removed. Again, where the penetration is thus suspended temporarily, the quick-

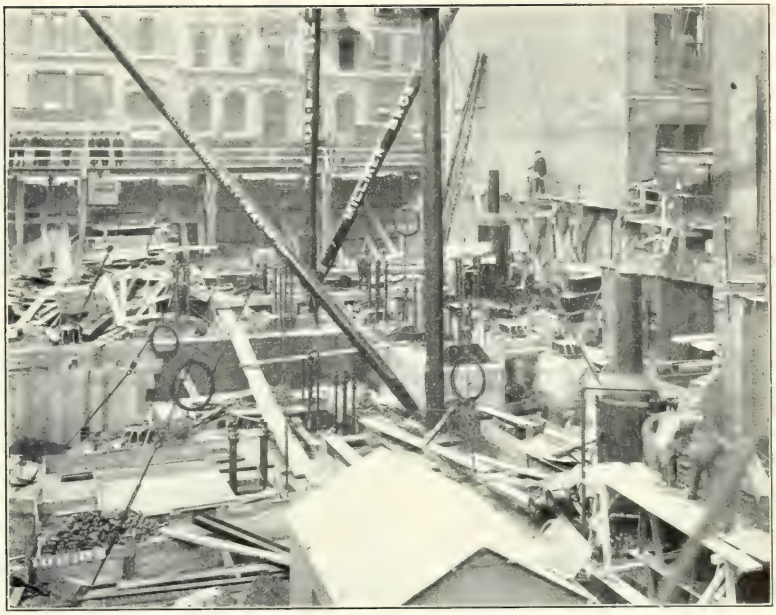


FIG. 1.—MARCH 25TH, 1907, TWO GUY DERRICKS READY TO ERECT STEELWORK.

FIG. 2.—APRIL 5TH, 1907, ERECTION OF STEELWORK WELL UNDER WAY.

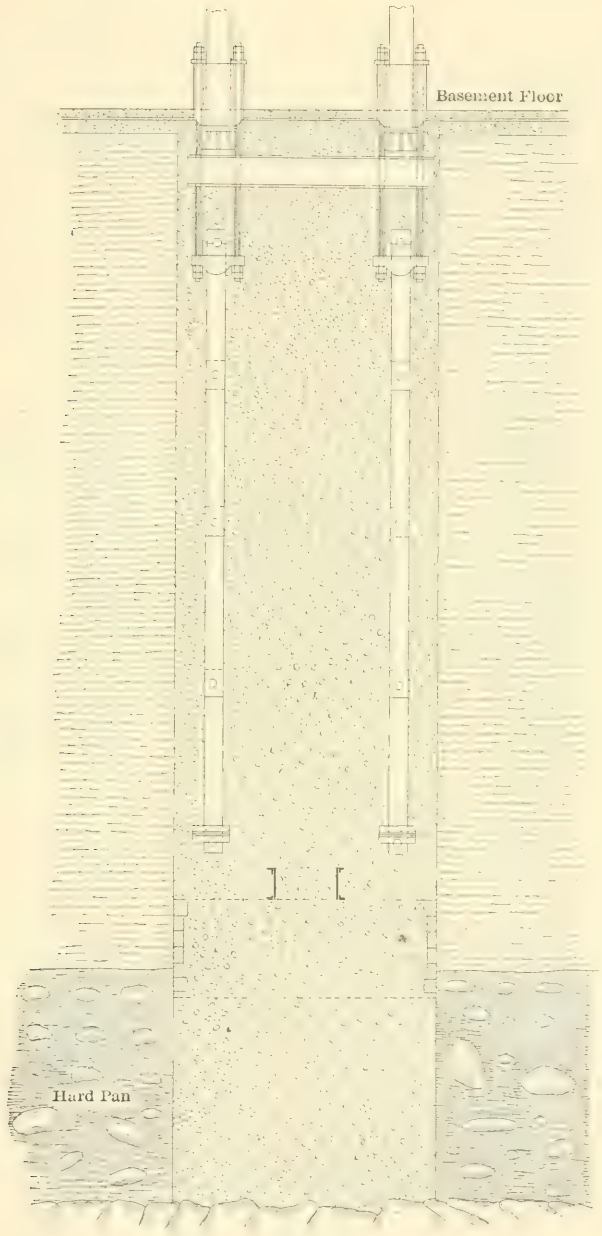


FIG. 11.

TABLE 1.—Loads on Column Bases, in Tons.

15 tons per sq. ft. allowed on caissons at top; no allowance for wind in caissons, except where wind strain exceeded dead and live load by 50 per cent.

Column number.	Dead load.	60% Live load.	Combined dead and live loads.	Wind loads.
1	756.7	56.4	813.1	747.1
2	614.3	98.5	712.8	747.1
3	447.5	108.9	556.4	373.5
4	447.5	108.9	556.4	375.5
5	786.2	88.4	874.6	747.1
6	884.2	42.6	926.8	747.1
7	763.0	79.9	842.9	747.1
8	289.2	131.6	420.8	747.1 Anchor.
9	284.4	129.6	414.0	373.5
10	284.4	129.6	414.0	373.5
11	289.2	131.6	420.8	747.1 Anchor.
12	763.0	79.9	842.9	747.1
13	451.1	94.9	546.0	373.5
14	284.4	129.6	414.0	373.5
15	327.4	147.4	474.8	747.1 Anchor.
16	327.4	147.4	474.8	747.1 Anchor.
17	284.4	129.6	414.0	373.5
18	445.4	92.0	537.4	373.5
19	528.7	92.6	621.3	373.5
20	284.4	129.6	414.0	373.5
21	327.4	147.4	474.8	747.1 Anchor.
22	327.4	147.4	474.8	747.1 Anchor.
23	284.4	129.6	414.0	373.5
24	454.7	94.3	549.0	373.5
25	602.5	91.8	694.3	747.1
26	289.2	131.6	420.8	747.1 Anchor.
27	284.4	129.6	414.0	747.1 Anchor.
28	284.4	129.6	414.0	747.1 Anchor.
29	289.2	131.6	420.8	747.1 Anchor.
30	602.5	91.8	694.3	747.1
31	596.4	68.3	664.7	650.3
32	614.3	98.5	712.8	747.1
33	447.5	108.9	556.4	373.5
34	447.5	108.9	556.4	373.5
35	614.3	98.5	712.8	747.1
36	596.4	68.3	664.7	747.1
37	73.2	46.4	119.5	.....
38	133.3	85.1	218.4	.....
39	191.1	121.8	312.9	.....
40	131.8	83.9	215.7	.....
41	272.2	25.7	297.9	.....
42	214.4	18.8	233.2	.....
43	214.4	18.8	233.2	.....
44	142.8	9.4	152.2	.....
45	120.3	27.2	147.5	.....
46	143.6	43.2	186.8	.....
47	143.6	43.2	186.8	.....
48	143.6	43.2	186.8	.....
49	120.3	27.2	147.5	.....
50	275.9	36.9	312.8	.....
51	350.0	52.0	402.0	.....
52	242.4	37.9	280.3	.....
53	242.4	37.9	280.3	.....
54	328.5	29.8	358.3	.....

TABLE 2.—RECORD OF SINKING CAISSONS FOR THE SINGER BUILDING, 1906-1907.

NUMBER OF CAISSON.	30	48-49	40-41	30-43	45-46	31-32	5-6	39	19-20	17-18	54	26-42	28-34	52-53	7-8
Order of sinking.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Size of caisson.	7′ 0″ × 7′ 2″	5′ 6″ × 16′ 4″	7′ 2″ × 11′ 10″	6′ 2″ × 16′ 4″	5′ 6″ × 16′ 4″	7′ 8″ × 18′ 4″	7′ 8″ × 21′ 2″	6′ 6″ diam.	5′ 6″ × 17′ 10″	5′ 6″ × 16′ 8″	7′ 0″ × 8′ 2″	6′ 2″ × 16′ 4″	6′ 2″ × 16′ 8″	5′ 6″ × 13′ 8″	7′ 8″ × 18′ 4″
Caisson arrived on lot. { Hour / Date.	5.30 P. M. Sept. 29	9.00 P. M. Oct. 11	4.30 P. M. Oct. 5	1.45 P. M. Oct. 6	4.30 P. M. Oct. 10	9.00 P. M. Oct. 6	6.30 P. M. Oct. 12	7.00 P. M. Oct. 26	9.30 P. M. Oct. 5	6.45 P. M. Oct. 9	5.30 P. M. Nov. 9	4.30 P. M. Oct. 8	11.25 A. M. Nov. 10	7.30 P. M. Nov. 13	8.00 P. M. Oct. 18
Compressed air on. { Hour / Date.	10.30 A. M. Oct. 15	1.30 P. M. Oct. 17	1.30 A. M. Oct. 16	5.00 P. M. Oct. 18	4.30 A. M. Oct. 27	1.00 A. M. Oct. 26	9.30 A. M. Oct. 29	6.00 P. M. Nov. 11	9.30 P. M. Oct. 31	9.00 P. M. Oct. 30	4.00 P. M. Nov. 16	12.30 P. M. Nov. 18	3.00 A. M. Nov. 18	7.00 A. M. Nov. 30	3.00 P. M. Nov. 12
Caisson reached hardpan. { Hour / Date.	2 A. M. Oct. 21	6.00 P. M. Oct. 21	10.00 A. M. Oct. 23	2.00 P. M. Oct. 27	9.30 A. M. Oct. 31	8.00 A. M. Nov. 4	10.30 A. M. Nov. 6	11.00 A. M. Nov. 13	10.00 P. M. Nov. 15	8.30 A. M. Oct. 31	7.00 P. M. Nov. 21	6.00 A. M. Nov. 23	11.00 P. M. Nov. 27	8.00 P. M. Nov. 30	1.30 A. M. Nov. 27
Excavation finished. { Hour / Date.	5.30 A. M. Oct. 22	9.00 P. M. Oct. 24	9.30 P. M. Oct. 24	2.00 P. M. Oct. 29	9.00 A. M. Nov. 2	6.00 A. M. Nov. 10	7.00 A. M. Nov. 11	11.00 A. M. Nov. 14	2.30 P. M. Nov. 19	7.30 P. M. Nov. 19	11.00 A. M. Nov. 23	11.30 A. M. Nov. 27	2.00 P. M. Dec. 1	2.00 P. M. Dec. 2	2.30 P. M. Dec. 2
Concreting begun in air-chamber { Hour / Date.	6 A. M. Oct. 23	10.15 P. M. Oct. 24	10.00 A. M. Oct. 25	2.30 P. M. Oct. 29	10.00 A. M. Nov. 2	7.00 A. M. Nov. 10	9.30 A. M. Nov. 13	12.30 P. M. Nov. 14	3.00 P. M. Nov. 19	8.45 P. M. Nov. 19	11.30 A. M. Nov. 23	12.30 P. M. Nov. 27	2.30 P. M. Dec. 1	2.30 A. M. Dec. 2	3.00 P. M. Dec. 2
Compressed air taken off. { Hour / Date.	4 A. M. Oct. 24	7.00 A. M. Oct. 26	12.30 P. M. Oct. 26	2.00 P. M. Oct. 30	10.00 A. M. Nov. 3	8.00 A. M. Nov. 11	4.00 P. M. Nov. 14	6.00 A. M. Nov. 15	4.00 P. M. Nov. 21	8.00 P. M. Nov. 21	11.59 P. M. Nov. 23	10.30 A. M. Nov. 29	3.30 P. M. Dec. 3	4.00 A. M. Dec. 4	1.30 P. M. Dec. 5
Number of hours under compressed air.	185½	209½	251	285	173½	391	390½	84	498½	527	176	382	372½	333	518½
Excavation stopped on	Hardpan	Rock	Hardpan	Hardpan	Hardpan	Rock	Rock	Hardpan	Rock	Rock	Hardpan	Rock	Rock	Hardpan	Rock
Ditching. { Feet. / Hours.	3 3	3 3	3 3	6 3	3 3	5 2	5 3	5 2	5 3	3 3	5 3	5 3	3 3	5 3	3 3
Excavation in quicksand. { Feet. / Hours	40 70	51 67	47 77	49 62	44 55	44 70	50½ 82	46 41	43 66	50½ 62	46½ 60	19½ 54	50¼ 88	45 57	42½ 77
Excavation in hardpan. { Feet.. / Hours }	10 25	13 69	11 35	7½ 31	11½ 36	25 142	19½ 117	82 24	10½ 82	30 91	12 40	21 102	16½ 87	14 42	21 132
Distance from curb to ground.	2′	15	15′	15′	18	18′	15	18	20′	15′	15′	15	15′	15′	20
" " " " top of concrete	18′ 6″	23′ 1″	18′ 8″	18′ 8″	23′ 1″	18′ 8″	27′ 2″	23′ 1″	18′ 8″	27′ 2″	18′ 8¾″	18′ 8½″	18′ 8″	17′ 5″	18′ 8″
" " " " " " haudpan	84′	71′ 4″	67′	69′	67′	67′	70′ 5″	69′	68′	70′ 6″	68′ 5″	69′ 6″	70′ 4″	64′ 11″	67′ 10½″
" " " " bottom of excavation.	74′ 6″	84′ 4″	77′ 11″	76′ 7″	78′ 6″	92′	89′ 11″	77′ 5″	84′ 7″	90′ 6	78′ 5″	90′ 6″	86′ 9′	78′ 11″	88′ 10½″
" " " " cutting edge	66′ 6″	73′	68′ 5″	71′ 4′	69′ 6″	68′ 6′	72′ 5″	71	69′ 1″	70′ 6′	68′ 5″	69′ 9	70′ 6″	67′ 11″	69′ 4½″
Number of feet of coffer-dam above concrete	0	0	4	4	0	0	14	0	0	14	4	4	7	4	4
" " cubic yards of excavation	99	244	197	229	213	385	449	78	254	256	135	282	273	177	358
" " " " concrete	101	263	189	213	183	378	377	72	237	215	124	265	259	171	364

TABLE 2.—RECORD OF SINKING CAISSONS FOR THE SINGER BUILDING, 1906–1907 (*Continued*).

Number of Caisson.	21-14	13-14	47	11-12	51	1-2	27-33	29-35	37-38	4-10	■	25-26	5-9	16-32	15-21
Order of sinking.	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Size of caisson.	5′ 6″ × 16′ 4″	5′ 6″ × 16′ 10″	6′ 6″ diam.	7′ 8″ × 18′ 2″	5′ 6″ × 11′ 8″	7′ 8″ × 19′ 4″	16′ 2″ × 16′ 8″	7′ 8″ × 17′ 8″	5′ 6″ × 16′ 2″	5′ 6″ × 16′ 8″	7′ 0″ × 7′ 0″	17′ 8″ × 17′ 4″	5′ 6″ × 16′ 8″	7′ 0″ × 17′ 0″	7′ 0″ × 17′ 0″
Caisson arrived on lot..... { Hour / Date.	4.00 P. M. Oct. 0	3.00 P. M. Oct. 23	7.00 P. M. Oct. 26	Noon Nov. 6	1.00 P. M. Nov. 28	11.45 A. M. Oct. 31	3.00 P. M. Dec. 6	4.00 P. M. Nov. 24	3.30 P. M. Dec. 5	2.00 P. M. Dec. 19	8.00 P. M. Dec. 24	8.45 P. M. Dec. 16	7.00 P. M. Dec. 19	6.00 P. M. Jan. 24	Noon Jan. 24
Compressed air on..... { Hour / Date.	8.00 A. M. Nov. 19	5.00 A. M. Nov. 21	8.40 P. M. Dec. 9	8.00 A. M. Dec. 3	6.30 P. M. Dec. 7	Noon Dec. 3	2.30 A. M. Dec. 14	10 A. M. Dec. 11	2 P. M. Dec. 15	3.00 P. M. Dec. 29	2.00 A. M. Jan. 4	12.30 P. M. Jan. 2	8.00 P. M. Dec. 30	Noon Feb. 5	12.30 A. M. Feb. 1
Caisson reached hardpan..... { Hour / Date.	8.00 A. M. Nov. 25	11.00 A. M. Dec. 6	8.00 A. M. Dec. 11	4.00 A. M. Dec. 7.	9.30 A. M. Dec. 15	7.30 A. M. Dec. 30	10.30 P. M. Dec. 23	6 P. M. Dec. 21	4.00 A. M. Jan. 4	4.00 P. M. Jan. 6	7.00 A. M. Jan. 10	11.30 A. M. Jan. 10	2.00 P. M. Jan. 12	6.00 P. M. Feb. 8	10.00 P. M. Feb. 14
Excavation finished..... { Hour / Date.	10.45 A. M. Dec. 8	10.00 P. M. Dec. 9	3.00 P. M. Dec. 12	7.00 A. M. Dec. 13	10.00 A. M. Dec. 17	11.00 P. M. Dec. 29	2.30 P. M. Dec. 30	7 A. M. Jan. 6	4.00 P. M. Jan. 6	2.45 P. M. Jan 9	7.00 A. M. Jan. 14	7.00 A. M. Jan 16	7.30 P. M. Jan. 15	11.30 P. M Feb. 11	8.30 A. M. Feb. 18
Concreting begun in air-chamber..... { Hour / Date.	11.30 A. M. Dec. 8	10.30 P. M. Dec. 9	3.30 P. M. Dec. 12	7.30 A. M. Dec. 13	10.30 A. M. Dec. 17	11.30 P. M. Dec. 29	3.00 P. M. Dec. 30	7.30 A. M. Jan. 6	4.30 P. M. Jan. 6	3.30 P. M. Jan. 9	7.30 A. M. Jan. 14	7.30 A. M. Jan. 16	8.30 P. M. Jan. 15	11.45 P. M. Feb. 11	9.00 A. M. Feb. 18
Compressed air taken off..... { Hour / Date.	5.00 A. M. Dec 10	1.30 P. M. Dec. 11	Noon Dec. 13	1.00 A. M. Dec. 15	5.30 A. M. Dec. 19	8.30 A. M. Dec. 29	12.30 P. M. Jan. 3	12.30 A. M. Jan. 9	11.30 P. M. Jan. 7	6.30 A. M. Jan. 11	10.30 A. M. Jan. 15	10.30 A. M. Jan. 18	9.30 A. M. Jan. 18	4.30 P. M. Feb. 13	10.15 P. M. Feb. 19.
Number of hours under compressed air.....	504	488½	87½	281	274½	620½	490	680½	561¼	363½	372¼	382	445½	190½	381¼
Excavation stopped on.....	Rock	Rock	Hardpan	Rock	Hardpan	Rock	Rock	Rock	Rock	Hardpan	Rock	Rock	Rock	Rock	Rock
Ditching ..... { Feet.. / Hours	5 3	5 3	5 2	5 3	5 3	5 3	5 3	5 3	5 3	5 3	5 2	5 3	5 3	5 3	5 3
Excavation in quicksand..... { Feet.. / Hours	52 58	42 56	52 85	49 70	49 52	48 75	50 77	50 96	43 58	51½ 57	49 43	41 64	51 58	50 74	50¾ 67
Excavation in hardpan..... { Feet.. / Hours	18.1 98	10 89	10 31	22 110	14½ 48	21 130	20 119	20 122	12 60	18 71	21¾ 96	20 139	18 67	19 78	17 84
Distance from curb to ground.....	15	20′	15′	15′	15	20′	15′	15′	20	15′	15′	20′	20′	15′	15′
" " " " top of concrete.....	27′ 2″	18′ 8″	23′ 1″	27′ 2″	19′ 5′	18′ 8″	18′ 8″	18′ 8″	18′ 8″	27′ 2″	27′ 2″	18′ 8″	23′ 1″	27′ 2″	18′ 8″
" " " " " hardpan.....	72′	67′	73′	69′	69′	67′	70′	70′ 2″	67′ 9″	71′ 6″	69′	66′	71′	69′ 9″	71′ 7″
" " " " bottom of excavation.	90′ 8″	86′ 1″	82′	90′ 8″	83′ 6″	87′ 8″	90′	90′ 2″	79′ 9″	89′ 2″	96′ 6″	91′ 11′	89′	89′ 9″	87′ 7″
" " " " cutting edge.....	73′ 9″	68′ 1″	73′ 6″	70′ 8″	70′ 0″	68′ 8	68′	72	65′ 9′	74′	70′ 3″	72′ 5″	73′ 3′	70′ 9″	72′ 7′
Number of feet of coffer-dam above concrete	14	4	12	18	4	4	10	10	4	10	14	0	14	14	8
" " cubic yards of excavation.....	577	226	82	390	183	371	285	327	197	233	137	284	251	329	300
" " " " concrete.....	309	228	82	327	152	377	271	358	199	212	114	358	234	275	304

sand is apt to pack against the caisson, greatly increasing the friction and requiring much more pig iron to overcome it.

This form of construction shows to the greatest advantage where it is possible to place all the concrete on the caisson before the sinking commences, or where the total depth of caisson and coffer-dam will not be more than 30 ft.

As soon as a caisson was placed and had some weight on it, it was "ditched," that is, sunk—generally about 5 ft.—without air, so as to make it safe to add the locks and more concrete and to make the bracing against overturning easier.

Instead of giving an average case or extreme cases, both of which are usually misleading, the history of every caisson is given in Table 2.

The writer has endeavored to confine this description to what was out of the ordinary, and the unique feature of the foundations of the Singer Building was certainly the undermining of Caisson No. 30-43. In tabulating No. 30-43, the results are those obtained when it was originally sunk, and do not include the tunneling.

Of course, a contractor always wants to stop pumping compressed air into a caisson as soon as he can do so with safety, after it has been filled with concrete; but it would be much better to keep up the air pressure for at least from 12 to 24 hours after the concreting is completed. The writer has repeatedly found that if this were done he could make the concrete in the air-chamber water-tight, whereas, if it were not done, the water would rapidly force its way through the concrete to the top, and nobody will claim that it is advisable to have water flowing through concrete before it has set.

It has been stated that the sample of concrete taken from the bottom of Caisson No. 30-43, when it was undermined, was of the hardest, showing that the concrete had set sufficiently before the air was taken off.

In Table 2, the caissons are designated by the number of the columns resting on them, as originally laid out on the column plan, Fig. 2.

The entire work was done under the personal direction of Mr. C. P. Coleman, the Executive Officer of the Singer Manufacturing Company, and Mr. Ernest Flagg, his Architect. Mr. O. F. Semsch was Chief Engineer for the Architect, and Mr. H. J. Howells had charge of the inspection for him. A. P. Boller and H. W. Hodge, Members, Am.

Soc. C. E., were the Consulting Engineers for the building, the Contractors being The Foundation Company, of which Mr. Franklin Remington is President, and D. E. Moran, M. Am. Soc. C. E., E. S. Jarrett, Assoc. M. Am. Soc. C. E., and L. L. Brown, M. Am. Soc. C. E., are members. Mr. Alexander Allaire was Superintendent in charge for the Contractors, and the writer was Consulting Engineer for the caisson work for the owners.

He desires to thank the owners, architects, engineers, and contractors for their kindness in furnishing information, records, etc., for this paper. It is unnecessary to state that their work on the building was well done.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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WATER-WORKS VALUATION AND FAIR RATES, IN THE LIGHT OF THE MAINE SUPREME COURT DECISIONS IN THE WATERVILLE AND BRUNSWICK CASES.*

By Leonard Metcalf, M. Am. Soc. C. E.

To be Presented December 2d, 1908.

Eleven years ago a timely paper by Wynkoop Kiersted, M. Am. Soc. C. E., upon the "Valuation of Water-Works Property,"† and the very interesting discussion which followed it, shed much light on this important question, which was the outgrowth of a well-defined movement toward municipal ownership of water-works.

Since that time many plants have been taken by the municipalities which they were serving, and some noteworthy legal battles have been fought under the leadership of lawyers of unusual ability, assisted by experts in this field of engineering. In the course of this litigation the courts have handed down some important decisions, from among which two opinions, covering the broad legal and economic principles

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

*Kennebec Water District *v.* City of Waterville and others, Savage, J., 97 Maine, 185, and Brunswick and Topsham Water District *v.* Maine Water Company, Savage, J., 99 Maine, 371–389.

†*Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 115–214.

underlying the subject of water-works valuation and rating, stand out in bold relief. These two opinions, rendered by the Supreme Court of Maine to guide two Boards of Appraisers in their valuation of certain water-works properties, were veritable beacons in the uncertain sea of public service corporation valuation and rating, and, with their application in three recent Maine cases and certain rulings in one of these—the Portland valuation case (by the tribunal upon which sat Judge Savage, the writer of the Maine opinions referred to, and one of his then Associate Justices, Judge Powers)—furnish the text for this paper.

The Waterville Opinion.

The opinion of Judge Savage, of the Supreme Court of Maine, in "Kennebec Water District *vs.* City of Waterville and others, Kennebec opinion, December 27, 1902," delivered in accordance with an act of the Legislature in advance of the taking, by the Kennebec Water District,* of the plant of the Maine Water Company (supplying the City of Waterville and the Towns of Winslow, Fairfield, and Benton), laid down for the first time instructions to appraisers—upon the law of eminent domain as applied to such cases, upon the considerations which should govern them in their determination of the value of the water-works plant taken, and upon damages, and upon evidence—instructions which may be said to have become classic, so often have they been quoted by counsel in water-works valuation cases, and so large a part have they played in the trial of such cases.

Ambiguity of Waterville Opinion on Interrelation of Value and Fair Rates.

Good and helpful as was the Waterville decision, however, it was so vague and ambiguous, in its discussion of the interrelation of value and fair rates, as to leave the student in an endless circle of reasoning and a labyrinth of doubt as to its meaning upon this point. It held that:

"The property to be taken, both plant and franchises, are to be appraised, having in view their value as property in itself and their value as a source of income. There are these elements of value, but only one value of one entire property is to be appraised in the end.

*See article of Harvey D. Eaton, Esq., of Waterville, *Journal*, New England Water-Works Assoc., Vol. XIX, pp. 147–162, June, 1905, "The Maine Water Districts and Appraisals."

These elements necessarily shade into each other." (97 Maine, 185, Par. 27.)

"In addition to structure value, the appraisers should allow just compensation for all the franchises, rights and privileges to be taken." (Par. 22.)

"In estimating even the structure value of the plant, allowance should be made for the fact, if proved, that the Company's water system is a going concern with a profitable business established, and with a present income assured and now being earned." (Par. 19.)

"The value of the franchise depends upon its net earning power, present and prospective, developed and capable of development, at reasonable rates; and the value to be assessed is the value to the seller and not to the buyer." (Par. 23.)

Note the last statement that "the value of the franchise depends upon its net earning power, * * * at reasonable rates * * *." As to the latter, Judge Savage says:

"The basis of all calculation as to the reasonableness of rates to be charged by a public service corporation is the fair value of the property used by it for the convenience of the public." (97 Maine, 185, Par. 3.)

"The * * * Water Company is a quasi-public, or public service corporation, and is entitled to charge reasonable rates for its services, and no more." (Par. 2.)

Thus Judge Savage appeared to hold that the franchise value depended upon the reasonableness of the rates, and that the reasonableness of the rates must depend (among other things) upon the franchise value. First, determine the fairness of the rates, and then the franchise value; but before one can determine the fairness of the rates, one must determine the franchise value (as the rates are to be based upon the entire value including franchise); and before one can determine the franchise value one must determine the equitableness of the rates. The very hypothesis begged the question. How could the mystic circle be broken?

In referring to this opinion as being, in the above-mentioned respect, "vague and ambiguous," the writer has used the words advisedly for the reason that Judge Savage did appear nevertheless to have in mind the fundamental gauge for the reasonableness of the rates, the basic principle outlined so beautifully in his opinion in the Brunswick case, as will be shown later in this paper. Thus, he says in the Waterville decision (97 Maine, 185):

"At the same time, the public have the right to demand that the rates shall be no higher than the services are worth to them, not in the aggregate, but as individuals." (Par. 4.)

"Summarized, these elemental principles are, the right of the company to derive a fair income based upon the fair value of the property at the time it is being used for the public, taking into account the cost of maintenance or depreciation, and current operating expenses, and the right of the public to have no more exacted than the services in themselves are worth." (Par. 5.)

Had Judge Savage but omitted the last seven words in his Paragraph 4 (the first just quoted), and thus made his gauge or standard the worth of the service to the public, or had he changed the last words in Paragraph 5 to read "than the services are worth to them, in the aggregate or as individuals," as he did in substance in his later opinion, all confusion would have been avoided.

Abstract of Waterville Opinion.

However, if the Waterville opinion was weak in this respect, it laid down specific rules for the appraisers' (and counsels' and experts') guidance upon the following points—rules which have been of great service in these cases—and thus propounded the law in advance of the establishment of the essential facts in the case to be adjudicated, a very difficult, exacting, and dangerous task:

The property of the Company includes the real estate or other property, the plant or physical system, all franchises, rights and privileges, exercised or capable of being exercised. (97 Maine, Par. 1, 27.)

The real estate must be appraised at its fair market value not at forced sale but at worth to seller under prudent and beneficial sale. (Par. 1, 7.)

Actual construction cost (with proper allowance for depreciation) and reproduction cost are competent but not conclusive or controlling evidence. (Par. 8.)

Structure value should include going concern with a profitable business established, and with a present income assured and now being earned, if proved. (Par. 19.) Good will should not be considered so far as the water system is practically exclusive. (Par. 20.)

In fixing structure value, including the element of going concern, consider also the present efficiency of the system, quality of the water and service, fitness of the plant and of the sources of water supply to meet reasonable requirements in present or future, necessary time to construct *de novo,* and the time and cost needed after construction to develop such new system to the level of the present one in respect to business

and income, and the added net income and profits which would accrue during this period of construction and development.* (Par. 11, 21.)

Franchise value depends upon its net earning power, under reasonable rates. The value to be assessed is the value to the seller, not the buyer, but always subject to the fairness of the rates. (Par 23.) So-called unlimited franchises are neither perpetual nor irrevocable (Par. 7), nevertheless the fact that the franchises are to be taken in no respect impairs their value. (Par. 26.) Past unfaithfulness, misbehavior or delinquencies of Company involving perhaps liability to forfeiture of the franchises are not a consideration. (Par. 13, 14.) In considering prospective development and use of the franchise, the further investment necessary and effect upon income and rates are pertinent matters. (Par. 24.)

The basis of all calculation as to the reasonableness of the rates is the fair value of the property used by the public service corporation for the convenience of the public, and the Company is entitled to charge reasonable rates for its service and no more. (Par. 3, 5, 2.) No more shall be exacted from the public than the services in themselves are worth, or are worth to the public as individuals. (Par. 4, 5.) The actual earnings of and rates charged by the Company are admissible evidence, but their weight is dependent upon the reasonableness of the rates among other things. (Par. 10.) The early hazard and losses are also to be considered with reference to the past income, which might otherwise have been excessive. (Par. 6.) Even if excessive, no deduction is to be made for the rates collected in the past, but the fact is to be borne in mind in considering the present or future income. (Par. 15.)

"Capitalization of income, even at reasonable rates, cannot be adopted as a sufficient or satisfactory test of present value. But while not a test, present and probable future earnings at reasonable rates are properly to be considered in determining the present value of the system."

The Brunswick Opinion.

In 1905 the Brunswick and Topsham Water District took the works of the Maine Water Company, supplying the City of Brunswick (leaving to the Company that portion of its plant supplying the City of Bath), and, as in the Waterville case, under legislative authority obtained from the Supreme Court of Maine, in advance of the trial of the case, an opinion covering the same general ground as the Waterville opinion—eminent domain, instruction to appraisers, franchise valuation, damages, evidence, and private and special laws.†

* The latter would seem to come rather under the head of franchise and future earnings than under structure value and going concern.

† Brunswick and Topsham Water District *v.* Maine Water Company. Savage, J., 99 Maine, 371–389.

Fortunately, this duty again fell to Judge Savage, for it gave him the opportunity to clear up the ambiguity of his earlier decision and to carry forward his admirable work—for these two masterly decisions bid fair to play a most important part in the future, not only in water-works valuation, but also in "fair-rates" cases involving public service corporations.

Abstract of Brunswick Opinion.

The legislature is not competent to prescribe rules for damage, but rules assented to by the inhabitants of the water district are made binding by their approval of the charter. (Maine, 99, p. 12.)

Actual construction cost bears upon reasonableness of rates and present value, but is not conclusive. If by rise of prices the present value is greater than the original cost, the owner is entitled to the appreciation; if less, he must suffer the loss. (Par. 4.)

Structure value must include consideration of the facts that the structure is in use, is a going concern, is lawfully in use, that is, that the structure is a going concern in use by virtue of a franchise. (Par. 2, 3.)

The reproduction cost is to be figured as of the date of taking at prices for materials and labor during a period long enough before that time for the construction and final completion of the works on the date of taking; and interest during the construction period is to be allowed upon the money invested as a part of the construction cost. (Par. 10.)

Reasonableness of the rates must be based upon the fair value of the property used by the company for the service of the public, including therein not only structure and going concern values, but also franchise value. (Par. 1.) The reasonableness of the rates relates to both the owner and the customer, but in case of conflict the rates must in any event be reasonable to the customer. (Par. 6.) The reasonableness of rates must be determined by the preponderance of evidence. (Par. 9.) The amount of money actually and wisely expended in producing the plant is a primary consideration. (Par. 5.) Profits which in the aggregate exceed a fair return on the property and franchise of the company do involve unreasonable rates. (Par. 8.)

Worth of the Service.—"A public service company cannot lawfully charge more than the services are reasonably worth to the public as individuals, even if charges so limited would fail to produce a fair return to the owner upon his property or investment." (Par. 7.)

"The worth of a water service in such connection is the worth to the customers as individuals, but as individuals making up a community of water takers." (Par. 14.)

"In estimating the value of a public service to the public or the customers, one of the elements necessary to be considered is the expense at which the public or customers, as a community, might serve themselves, were they free to do so, and were it not for the practically exclusive franchises of the supplying company. Water is to be regarded as a product, and the cost at which it can be produced or distributed is an important, though not the only, element of its worth." (Par. 13.)

Communities are entitled to the benefit of natural advantages, and the company is not entitled to charge enhanced rates based upon the cost of using the more expensive source. (Par. 15.)

Damages for severance are to be allowed. (Par. 11, 16.)

Eminent domain. The award is to be made under the principles of eminent domain and upon such principles of eminent domain as were agreed to by the voting constituents of the water district by approving the charter.

General Considerations Affecting Water-Works Valuation and Fair Rates.

The following outline indicates in a general way the more important considerations bearing upon water-works valuation and rating and the scope of the inquiry which the expert will be likely to wish to make in order to form his judgment as to the value of any particular works or the fairness of the rates charged by them:

A.—Data Relating to Community.—

1.—Character of population, and industries and buildings.

2.—Population: Past, present, and probable future, United States, State, and Local Census.

3.—Assessed valuation: Past and present.

B.—History of Plant and Dates of Construction and Further Development.

C.—Legal Rights.—

1.—Form of incorporation: Charter.

2.—Franchise: Exclusive or not, perpetual or limited, terms of purchase or extension.

3.—Contracts for public and private water service: Terms, time, and prices.

4.—Tax exemption or otherwise.

5.—General laws affecting company: Local ordinances or orders.

D.—Physical Plant.—

1.—Source of supply: Character, artesian, ground, surface.
2.—Population on water-sheds, and sanitary conditions.
3.—Quality.
4.—Quantity, available drainage area; storage and pondage, variation in yield.
5.—Adequacy for present and probable future needs.
6.—Local rainfall and flow statistics.
7.—Number of taps, services, and consumers; number of meters.
8.—Water consumption and uses: Gallons, average per day, total, per capita, per service, or tap.
9.—Variation in and character of consumption.
10.—Reservoir system: Character, amount, location or distribution.
11.—Pipe distribution system: Character, kind, and extent.
12.—Efficiency: Age, deterioration, probable life, effects of electrolysis, etc.
13.—Available domestic supply, quantity and pressure.
14.—Available fire supply, number and character of streams.
15.—Adequacy of service as compared with present and probable future needs.
16.—Necessary re-enforcement.
17.—Possible competitive sources of supply, with data for comparative estimates of cost.
18.—Pumping station data.
19.—Purification data, sedimentation, filtration, softening, etc.
20.—Plans: General and detail; elevations, etc.

E.—Local Prices of Labor and Building Materials; Hours of Employment; Ordinances in Regard to Labor, etc.

F.—Rates.—

1.—Schedule of public, commercial, and domestic rates, past and present.
2.—Rates elsewhere in vicinity (probably not admissible as evidence, but of service to experts).

G.—Operating Statistics.—(Annual for period of years, or during whole life of plant, if possible.)

1.—Gross income.

a.—Classification of gross income: Public hydrants and other uses; domestic and manufacturing; metered and fixture service; license or permit for tapping; profit on services; house plumbing, etc.

2.—Operating expenses.

a.—Administration salaries and wages; legal, engineering, general, and miscellaneous expense.

b.—Buildings and real estate: Rental, care and repairs.

c.—Pumping: Labor, fuel, oil and waste, sundries, repairs.

d.—Filtration: Labor, chemical coagulation supplies, sundries, repairs.

e.—Street mains and service pipe work: Labor, supplies, miscellaneous repairs.

f.—Meters: Labor, supplies, miscellaneous repairs.

g.—Insurance and taxes.

3.—Net income = gross income less operating expenses.

4.—Maintenance = depreciation and renewals.

5.—Fixed charges: Interest on bonds and floating loans.

6.—Sinking fund requirements.

7.—Dividends on stock and surplus.

8.—Unit figures: Gross and net income, per million gallons daily, per tap, per capita, per mile of pipe, to be derived from the above.

H.—Sinking Fund, Surplus, and Interest and Dividends Paid in Past, and Resulting Average Earnings upon Original Cost or Investment.

I.—Real Estate and Water Rights, etc.—

1.—Cost of.

2.—Market value of.

3.—Assessed value of.

J.—Structure Value.—

1.—Original cost of original plant and subsequent extensions and betterments:

a.—Date and amount of.

b.—Character of prices, as compared with normal prices of that time.

c.—Reasonableness of outlays and provisions for the future.

d.—Date and amount of extensions and betterments.

2.—Reproduction cost as of date of taking, buying materials sufficiently in advance to have plant ready for operation on date of taking.

3.—Depreciation: Annual and total to date.

4.—Cost of betterments or necessary improvement of service, completed as of date of taking.

a.—Anticipated increase in earnings from these betterments.

b.—Anticipated increase or decrease in operating expenses from these betterments.

c.—Depreciation and fixed charges upon these betterments.

d.—Difference, or net increment or charge.

5.—Competitive sources. Cost of and charges that would result.

K.—Lawful Rates and Value.—

1.—Reasonable ratio of profit to the owner or promoter in excess of the actual reasonable cost of the property. This must be limited by the consideration of the profit or excess over cost which the public would probably have been willing to allow the corporation at the time of its incorporation, as compensation for relieving the city of the necessity of undertaking the work itself, as it might have done at cost, but which it then elected to have done for it at cost plus profit. (The reasonable amount of this profit will be discussed hereafter.)

2.—Rate of interest to be allowed upon investment, or property, commensurate with property of equal security, safety, and risk.

3.—Annual allowance for depreciation.

4.—Market value of real estate and supplies on hand.

5.—Worth of service to consumer or lawful gross annual income for given investment. Reasonable annual operating cost including repairs; plus annual depreciation: plus interest upon the sum of the reproduction cost* and profit increment less total depreciation to date (if the

* The reasonableness of using a fair average reproduction cost instead of the reproduction cost upon the date of taking, or the original cost, is discussed later.

latter has been previously earned). This sum measures the lawful annual gross income, if the works are capable of earning it.

6.—Comparison of lawful with actual annual gross income.

7.—Value determined by reproduction cost plus profit increment less total depreciation to date (if previously earned), plus market value of real estate and supplies on hand, provided the actual annual gross income equals or exceeds the lawful annual gross income.

8.—If actual annual gross income is less than the above determined lawful annual gross income, value is determined by capitalizing, at the assumed fair rate of interest, the actual net income derived by deducting from the gross annual income the operating expenses and annual depreciation allowance; increasing this sum by the market value of the real estate, and decreasing it by the depreciation to date (if the latter has been earned previously in the rates).

9.—Calculate equivalent annual rate of return to the owner of this lawful income, upon the actual cost and upon the fair average reproduction cost of the works.

10.—Corrections to be applied on account of necessary betterment or improvement in efficiency of the plant, with due allowance for any possible increase as well as decrease in income, operating expenses, maintenance and fixed charges.

11.—Resulting value to owner, subject to limitation of reasonable rates and reasonably efficient service (including therein the necessary improvements or betterments).

L.—Results to City of Award of Assumed Value.—

1.—Assumed value or award.

2.—Annual combined cost of operation, depreciation and (or) sinking fund, and interest upon investment.

3.—Present annual income.

4.—Net result upon present conditions.

5.—Similar figures with allowance for increase or decrease in earnings, operating expenses, depreciation and fixed charges, due to betterments or necessary improvement of

efficiency or standard of service. It may be of interest, though not logically correct, to include in the total investment the actual cost to the city of acquiring the property, that is, the cost of court, legal, expert, and witness fees and expenses.

M.—Going Concern Element of Value.—Note that this element of value is included in the above valuation, and is of significance or importance chiefly in cases involving valuation of plants taken after expiration of their franchises.

Company's Plant.	*Newly Built Plant.*
First Year.—Net income at date of taking.	Probable gross income for year, less operating expenses, plus saving in state and county taxes.*
Loss in first year by operation of new plant by city.	
Second year.—Net annual income as of date of taking.	Probable gross income for year, less operating expenses, plus saving in state and county taxes.*
Loss in second year by operation of new plant by city.	

Similarly for third, fourth, etc., years to time at which the city may be assumed to have developed the business to a point equal to the net annual income at date of taking.

The total sum of the present worths of these annual losses may be considered to be the "going concern value."

N.—The Subjoined Form for Collecting and Preserving Financial Data Relating to Water-Works.—This form (page 1113), used by the writer for some years, may also be of service.

Some Comments Upon the Above Considerations Affecting Water-Works Valuation and Rates.

Legal Rights.

Inasmuch as the majority of Maine water-works franchises may be classed under the popular head of "perpetual franchises," and as

* Interest charges not included as they are the same for both plants, if we assume both plants ready for operation as of date of taking. Annual depreciation assumed to be the same for both plants, hence ignored. The town taxes are not assumed to be saved by the town, as it must make good, by general taxation, any loss from reduction of taxes upon Company's plant or abolition thereof in case of taking of the plant by the city.

WATER-WORKS STATISTICS.

Place_____

Year									
Population									
Grav. or Pumpg									
Miles of Pipe									
Pop. per mi. of pipe									
Range of Sizes									
% Wrought-Iron									
Taps in service									
Per 1000 popul.									
Per mi. of pipe									
Meters									
% of taps metered									
Consump. annual m. g.									
Daily g. d.									
Per capita g. d.									
Per mi. of pipe g. d.									
Per tap g. d.									
Pressure (range) lbs.									
Hydrants-number									
Per 1000 popul.									
Per mi. of pipe									
Bonds									
Rate of Interest									
Capital Stock									
Floating Debt									
Gross Income Total									
Per capita									
Per mi. of pipe									
Per tap									
Per m. g. consump.									
Gross Inc. excl. Hyds.									
Per capita									
Per mi. of pipe									
Per tap									
Per m. g.									
Net Inc. excl. interest									
% of gross									
Per capita									
Per mi. of pipe									
Per tap									
Per mil. gals.									
Hydrant Rental									
Per hydrant									
Per mi. of pipe									
Taxes, annual									
Per $1000 bonds									
Cost of operation									
Per capita									
Per tap									
Per mi. of pipe									
Per mil. gals.									
% of gross income									

the illustrations given in this paper come under this class, the discussion has been limited to franchises of this class. The general principle of the worth of the service to the consumer is equally applicable and valuable, however, in its bearing upon fair rates, to limited term franchises.

Original Cost.

The Brunswick opinion gives new life and significance to the original cost, which, heretofore, has received comparatively little consideration, if any, in valuation proceedings in the United States, other than in its remote bearing upon the total depreciation of the plant to date. While it is recognized that in many, perhaps in most, cases it may not be possible to determine the actual first cost, where available, it may have a very important bearing upon the case, particularly if the cost was a normal and reasonable one at the date of construction.

The reasonable annual return to be allowed the builder of the works upon their original cost (referred to later) is largely a matter of judgment, and will be influenced by many considerations. Roughly speaking, it may vary from 7 to 10% or even more, the smaller rate reflecting the larger works, the better topographical or physical conditions, the more promising field, the greater ease and lesser expense of making provision for the future, etc., the greater freedom from danger of competition, the smaller hazard, and the more stable character of the community; the larger rate reflecting the reverse.

Reproduction Cost.

This cost has always been recognized as pertinent and important, though not controlling, in valuation cases.

It has had its greatest influence in the case of works taken after expiration of franchise, when the reproduction cost, depreciation, and going concern value have been very important, if not controlling, elements of value.

Even so, under the Brunswick ruling, reproduction cost has a new force in its bearing upon the cost to the public of supplying itself—the worth of the service to the consumer—and hence upon the reasonableness of the rates. Upon the latter subject more will be said under the caption of the "Measure of Fair Rates."

Reproduction Cost.—Court Rulings upon Prices to be Assumed.—The method of figuring the reproduction cost of the plant, prescribed

by the courts in different States, varies considerably, so that the results obtained under one ruling may be quite different from those obtained under another, particularly in large plants involving a construction period of several years.

The Maine opinions clearly prescribe the assumption of prices prevailing at a time in advance of or prior to the date of taking, corresponding to the probable period of construction, upon the theory that the plant is to be built and delivered complete upon the actual date of taking.

In the Kansas City case,* and in a number of other cases, on the other hand, prices prevailing upon the date of the taking were taken, upon the theory that the valuation was made as of the date of taking and hence that the prices of that date should be assumed.

Thus, in the Portland (Maine) valuation suit, the several engineers assumed periods of construction from 2 to 3 years, and made allowance for the interest charges upon construction within this period, with a resulting agreed reproduction cost, including hydrants (owned by the city) and services (laid at the expense of the water takers), of $2 853 696, cast-iron pipe being figured at approximately $29 per net ton—all under the Maine ruling before referred to. Had the Kansas City ruling been used, the price of cast-iron pipe would have been approximately $25, with a reduction in the reproduction cost of about $80 000. While the deduction for depreciation would have offset this decrease in some measure, the net results would still have been much less.

This illustration leads to the natural suggestion that in equity, at least, a reproduction cost based upon normal or average prices for a period of years prior to date of taking would be a fairer criterion than one based upon the monthly or daily fluctuations of the material and labor market.†

Reproduction Cost.—Identical Structure vs. Structure of Equal Efficiency.—The courts are also divided upon the subject as to whether, in calculating the reproduction cost of a structure, the actual lines of the structure or, in other words, the identical structure under consideration should be adhered to, or, on the other hand, whether the cost may be measured by that of a structure of equal efficiency. The

* National Water-Works Company *vs.* Kansas City, 62 Federal Reporter, 853-869.
† See also pages 1135, 1136, 1139 *et seq.*

Maine Courts have taken the former view, the Federal Court in the Kansas City suit, the latter. In a comparatively new or young plant the difference may be insignificant, in an old plant it is likely to be large. Something may be said in favor of both methods, though the latter, if reasonably applied, would appear to have greater merit from a business standpoint. The owner may claim that the plant, though obsolete or old-fashioned and not what he would build were he building anew on the date of taking, is still serving the public adequately; whereas the buyer of the plant would not be likely to value a multiplicity of parts, as, for instance, three or four supply mains, built in sequence and costing far more than one unit, at a materially higher figure than one equivalent unit, except perhaps in so far as duplication may give an added factor of safety in operation. Moreover, if, through the agency of the rates, the owner of the property has collected from the public a depreciation allowance sufficient to make good this loss in efficiency, it is but just to the public and not unfair to him to consider reproduction cost from the point of view of a structure of equal efficiency rather than of identical parts. Whatever the views of the expert, however, the laws of the community or the counsels' interpretation of them will hold.

Depreciation.

A water-works system has a limited life or period of usefulness. The structure suffers depreciation with the lapse of time. The various elements composing the physical plant have to be removed and replaced from time to time. Broadly speaking, this depreciation may be classified under physical and functional depreciation, and, in certain cases, perhaps, under a third class growing out of expediency or extraordinary external requirements. By physical depreciation is meant the actual wear and tear of operation upon the structure; by functional depreciation, the obsolescence of the structure or its inability, for one reason or another, effectively to meet throughout its life the full requirements of the service. Thus the structure may suffer total depreciation and be thrown out of service, not only because through wear and tear it has reached a condition where further expenditures for repairs or attempts to make it suitable for the required service would not be economical or expedient, but also because recent improvements, or new inventions, new developments and radical changes in service, or

the demands of one kind or another involving sweeping changes in the existing plant, make abandonment necessary.

The rates of depreciation of the different component parts of a water-works system vary greatly, both in the final limit of useful life and in the progress or rate of depreciation during that life. The depreciation may be at a uniform rate, or at a variable rate, or indeed at an irregular rate, depending upon various factors. Thus in the case of a reservoir, the structure may suffer substantially no depreciation until the time when it has to be abandoned by the requirements of larger size, greater pressure, or other detail of service; when this time does come, if the reservoir is actually abandoned, it suffers total depreciation, or, if it continues to serve a partial need, partial depreciation. A pumping engine may suffer virtually no depreciation in the first year of its life, indeed it may run more smoothly than when new; thereafter, it may depreciate steadily, if slightly, until it is outgrown and another larger machine has to be added, when it suddenly suffers a very large depreciation; it may then be used as an auxiliary machine for a portion of the time for a further period of years, until even this limited use proves uneconomical, though the engine may be held for emergency use for some time until it is finally abandoned, and only junk value less cost of removal remains. Clearly such a life history may be a very erratic one.

Three general methods of calculating the total depreciation of a water-works plant at any time in its life history, have been developed: First, the sinking-fund method; second, the constant-rate or straight-line method; and, third, what might be termed the individual analysis method, by consideration of the local conditions, more particularly with their reference to one another and their bearing upon the general efficiency of the plant.

Allen Hazen, M. Am. Soc. C. E. (and perhaps others) has suggested that in calculating the depreciation of cast-iron pipe, a consideration of the loss in carrying capacity would furnish an added criterion. It certainly does, although its direct application is impossible, for the reason that the pipe system is designed with some regard for this very factor—gradual loss in carrying capacity—hence, to charge a greater depreciation in the early life of the plant in addition to the burden of the greater pipe sizes used to meet this very contingency, would be to make a double charge against earnings. By

this criterion the loss in the early years of the life of the pipes would exceed that resulting from the application of the sinking-fund method. The true results may lie somewhere between the two, though if the assumed life of the structure in the sinking-fund method be moderate, any allowance on this score may be excessive.

In the opinion of the writer, all these methods are but aids to judgment, and must be applied with care and discretion, and with a clear conception of the extent of the modifying influences or considerations; no subject in the engineering field requires richer experience or riper judgment. Obviously, the first and third or second and third methods may give identical results. The first method shows a smaller rate of depreciation in the earlier years, and a larger rate of depreciation in the latter years, of the life history of the plant under consideration, than does the second method. In valuation the third method can be applied as readily as the others, but in accounting—in the operation of a water-works plant—the first two methods are the more specific, easier and more certain of application. In operation it is generally more satisfactory to the stockholders to have an agreed and known percentage of first cost or of income charged off to depreciation annually, than to have lump sum amounts charged off as the service demands and the annual income or surplus may warrant. The former method of distribution should be no less accurate in a period of years, and should reduce the extremes of fluctuation in dividends. These general methods have been described so clearly elsewhere in the articles referred to herein, as to make extended comment unnecessary.

It may be said, however, that the work involved in calculating depreciation by the sinking-fund method is nominal when the probable useful life of the structure and rate of interest for the sinking fund have been determined, provided sinking fund tables are available.

Bond Formulas, Compound Interest, and Sinking Funds.—For the convenience of those who may have occasion to figure bond yields, interest, or sinking funds, when tables are not to be had, the following formulas are presented. They were collated, arranged, and annotated for the writer several years ago by one of his assistant engineers, Frank H. Carter, Assoc. M. Am. Soc. C. E. The authority for and reference to the formula is given in each case.

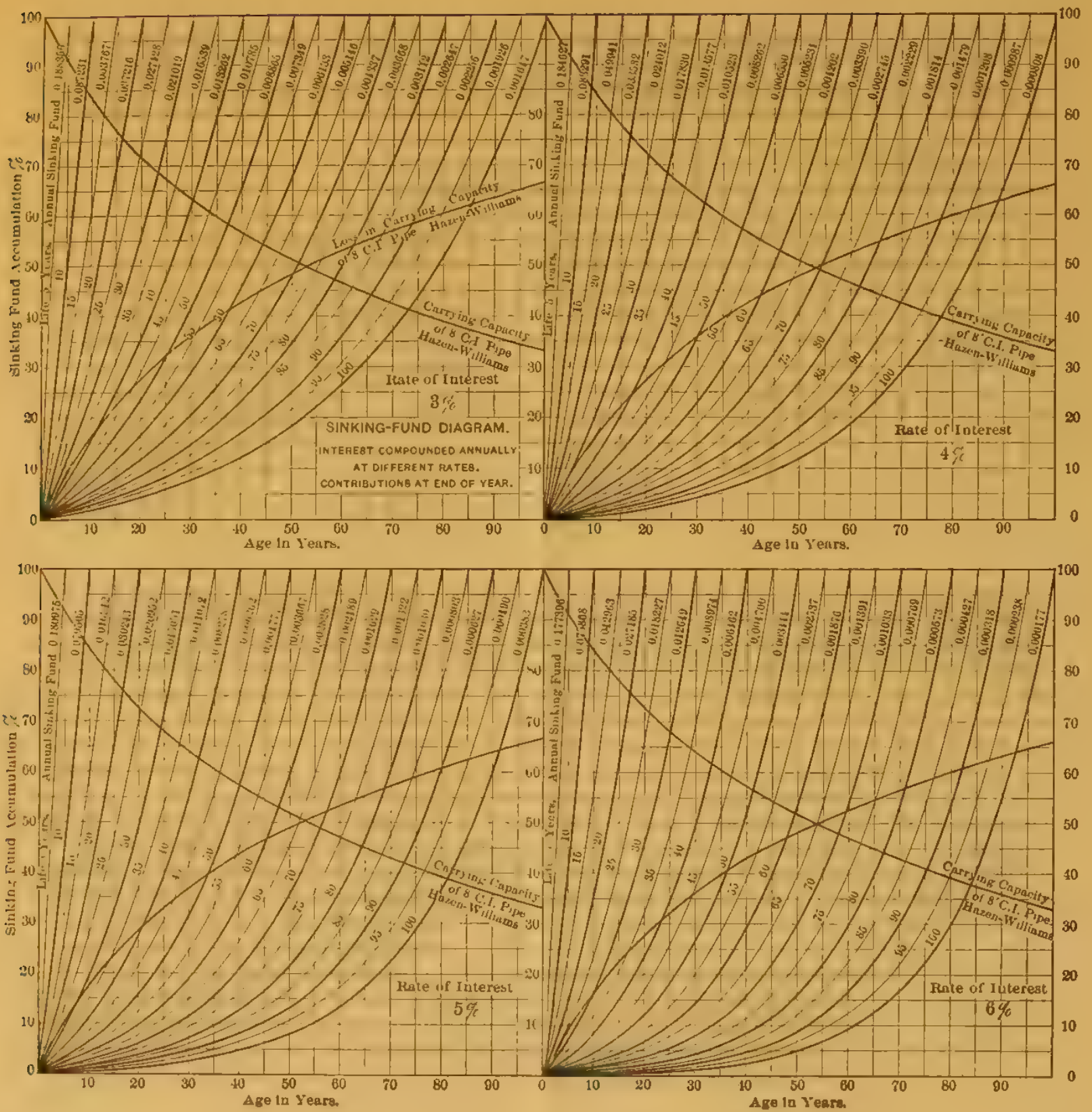
SINKING-FUND DIAGRAM.
INTEREST COMPOUNDED ANNUALLY AT DIFFERENT RATES.
CONTRIBUTIONS AT END OF YEAR.
Rate of Interest 3%
Rate of Interest 4%
Rate of Interest 5%
Rate of Interest 6%
Sinking Fund Accumulation %
Annual Sinking Fund
Life in Years.
Loss in Carrying Capacity of 8" C.I. Pipe Hazen-Williams
Carrying Capacity of 8" C.I. Pipe Hazen-Williams
Age in Years.

TABLE 1.—Sinking-Fund Data.*

Life of structure.	Annual contribution to sinking-fund in percentage of first cost, with different assumed interest rates upon sinking-fund accumulations.			
Years.	3%	4%	5%	6%
5	18.8355%	18.4627%	18.0975%	17.7396%
10	8.7231	8.3291	7.9505	7.5868
15	5.3767	4.9941	4.6342	4.2963
20	3.7216	3.3582	3.0243	2.7185
25	2.7428	2.4012	2.0952	1.8227
30	2.1019	1.7830	1.5051	1.2649
35	1.6539	1.3577	1.1072	0.8974
40	1.3262	1.0523	0.8278	0.6462
45	1.0785	0.8262	0.6262	0.4700
50	0.8865	0.6450	0.4777	0.3444
55	0.7349	0.5231	0.3667	0.2537
60	0.6133	0.4202	0.2828	0.1876
65	0.5146	0.3390	0.2189	0.1391
70	0.4337	0.2745	0.1699	0.1033
75	0.3668	0.2229	0.1322	0.0769
80	0.3112	0.1814	0.1030	0.0573
85	0.2647	0.1479	0.0803	0.0427
90	0.2256	0.1208	0.0627	0.0318
95	0.1926	0.0987	0.0490	0.0238
100	0.1647	0.0808	0.0383	0.0177

Note.—Deposits at end of year compounded annually.
*Abstracted from "Robinsonian Bond and Investment Tables," J. Watts Robinson.

TABLE 2.—Sinking-Fund Data.*

Annual contribution to sinking-fund.	Approximate age at maturity or life of structure, with different assumed interest rates upon sinking-fund accumulations.			
%	3%	4%	5%	6%
¼	87 yrs.	72 yrs.	62 yrs.	55 yrs.
⅜	74 "	63 "	55 "	49 "
½	66 "	56 "	49 "	44 "
⅝	59 "	51 "	45 "	41 "
¾	54 "	47 "	42 "	38 "
⅞	50 "	44 "	39 "	35 "
1	47 "	41 "	37 "	33 "
1¼	41 "	37 "	33 "	30 "
1½	37 "	33 "	30 "	28 "
1¾	34 "	30 "	28 "	26 "
2	31 "	28 "	26 "	24 "
2½	27 "	24 "	23 "	21 "
3	23 "	22 "	20 "	19 "
3½	21 "	19 "	18 "	17 "
4	19 "	17 "	17 "	16 "
5	16 "	15 "	14 "	14 "
6	14 "	13 "	12 "	12 "
7	12 "	12 "	11 "	11 "
8	11 "	10 "	10 "	10 "
9	10 "	9 "	9 "	9 "
10	9 "	9 "	8 "	8 "
15	6 "	6 "	6 "	6 "
20	5 "	5 "	5 "	5 "

Note—Deposits at end of year compounded annually.
*Abstracted from "Robinsonian Bond and Investment Tables," J. Watts Robinson.

Bonds. B = face of bond,

R' = amount of $1 for 1 yr. @ r' rate,

P = price of bond,

n = years bond has yet to run,

r = rate of interest bond bears,

r' = rate of interest at which coupons are reinvested,

x = rate of interest which investment yields.

All rates of interest are here taken per year.

Common Method.—

$$P(1+x)^n = Br(1+r')^{n-1} + Br(1+r')^{n-2} + \ldots Br + B$$

$$= B + \frac{Br[(1+r')^n - 1]}{r'}$$

$$\therefore 1 + x = \left(\frac{B}{P} + \frac{Br[(1+r')^n - 1]}{Pr'}\right)^{\frac{1}{n}}$$

$$1 + x = \left(\frac{Br' + Br(1+r')^n - Br}{Pr'}\right)^{\frac{1}{n}} \quad \ldots\ldots\ldots\ldots (1)$$

(Wentworth's Algebra.)

The Robinsonian tables are not figured on the basis of the above formula, but as follows:*

Robinsonian Bond Values.

$$(Br - Pr)\left(\frac{(R')^n - 1}{r'}\right) + B = P$$

$$Br(R')^n - Br - \frac{Pr((R')^n - 1)}{r'} + Br' = Pr'$$

$$\therefore P = \frac{B\left(r\left[(R')^n - 1\right] + r'\right)}{r' + x\left((R')^n - 1\right)} \quad \ldots\ldots\ldots\ldots (5)$$

(After Robinson.)

or, as J. Watts Robinson assumes r' at 4% :

$$P = \frac{B\left(r\left[1.04^n - 1\right] + 0.04\right)}{0.04 + x(1.04^n - 1)} \quad \ldots\ldots\ldots\ldots (2)$$

$$x = \frac{0.04(B - P) + Br(1.04^n - 1)}{P(1.04^n - 1)} \quad \ldots\ldots\ldots\ldots (3)$$

$$\text{or } x = \frac{r'(B - P) + Br\left[(1 + r')^n - 1\right]}{P\left[(1 + r')^n - 1\right]} \quad \ldots\ldots\ldots\ldots (4)$$

(After Robinson.)

* See page 1122.

In the Robinsonian tables interest is compounded semi-annually.

Formulas 2 and 3 differ from all other bond value formulas in this: that the values are computed on the hypothesis that the reinvestments of interest necessary to compensate for the premium or discount at which a bond is bought are here invariably made at the rate of 4% per annum.

Compound Interest and Sinking Fund Formulas.—

B = face of bond,

P = principal, or present worth,

r = rate of interest on one dollar for one year,

R = amount of one dollar for one year,

n = number of years,

A = amount of P for n years,

q = number of times in year that interest is payable,

a_1 = annuity payable at beginning of year,

a_2 = annuity payable at end of year,

a_3 = annuity payable at the beginning of each year and at the end of last year,

S = amount of sinking fund at end of n years.

$A = P\left(1 + \frac{r}{q}\right)^{qn}$; or if $q = 1 \therefore A = PR^n$. Compound interest. (Wentworth's Algebra. p. 377.)

$P = \frac{A}{\left(1 + \frac{r}{q}\right)^{qn}}$; or if $q = 1 \therefore P = \frac{A}{R^n}$. Present worth.....(6) (Wentworth's Algebra, p. 377.)

$P = \frac{a_1 (R^{n+1} - R)}{R^n (R - 1)}$. Present worth of annuity payable at beginning of year.....(7) (F. H. C.)

$a_1 = \frac{P R^n (R - 1)}{R^{n+1} - R}$. Annuity payable at beginning of year.....(8) (F. H. C.)

$P = \frac{a_2 (R^n - 1)}{R^n (R - 1)}$. Present worth of annuity payable at end of year.....(9) (Well's Algebra, p. 381.)

$a_2 = \frac{P R^n (R - 1)}{R^n - 1}$. Annuity payable at end of year.....(10) (Well's Algebra, p. 381.) (Wentworth's Algebra, p. 379.)

$S = \frac{a_1 (R^{n+1} - R)}{R - 1}$. Sinking fund, payment at beginning of year..(11) (After Frizell.)

$$a_1 = \frac{S\,(R-1}{R^{n+1}-R}.$$ Annual payment made at beginning of year to establish sinking fund.....(12)
(After Frizell.)

$$S = \frac{a_2\,(R^n-1)}{R-1}.$$ Sinking fund, payment at end of year.....(13)
(After Frizell, p. 94.)

$$a_2 = \frac{S\,(R-1)}{R^n-1}.$$ Annual payment made at end of year to establish sinking fund.....(14)
(After Frizell, p. 94.)

$$P = \frac{a_2}{R-1}.$$ Present worth. Perpetual annuity.....(15)
(Well's Algebra.)

$$P = \frac{a_2\,(R^n-1)}{R^{m+n}\,(R-1)}.$$ Present worth of an annuity to begin after m years and to continue for n years.....(16)
(Well's Algebra.)

$$P = \frac{a_2}{R^m\,(R-1)}.$$ Present worth of perpetual annuity to begin after m years.....(17)
(Well's Algebra.)

$$P = \frac{a_3\,(R^{n+1}-1)}{R^n\,(R-1)}.$$ Present worth annuity.....(18)

$$a_3 = \frac{PR^n\,(R-1)}{R^{n+1}-1}.$$ Annuity from P.....(19)

$$S = \frac{a_3\,(R^{n+1}-1)}{R-1}.$$ Sinking fund.....(20)

$$a^3 = \frac{S\,(R-1)}{R^{n+1}-1}.$$ Annual payment to establish sinking fund in n years with $n+1$ payments.....(21)
(After Robinsonian Tables.)

The annual payments to be made at the beginning of each year and at the end of the last year, or $n+1$ payments will be made.

Method of Comparing Economy of Structures.—*

A = first cost,

X = repairs at intervals of s years,

n = life of structure,

D = value at which plant may be sold at end of n years,

C = capital involved at present,

r = rate of interest,

q = number of times in 1 year that interest is payable.

$$C = A + \frac{X}{\left[\left(1+\frac{r}{q}\right)^{qs}-1\right]} + \frac{A-D}{\left[\left(1+\frac{r}{q}\right)^{qn}-1\right]} \quad \text{.....(22)}$$

*G. F. Swain, M. Am. Soc. C. E., *Journal*, N. E. W. W. Vol. II, March, 1888, No. 3, p. 32.

Proof of Formula 22:

By Formula 15,

$$P = \frac{a_2}{R - 1}, \text{ and } a_2 = P(R - 1)$$

By Formula 13,

$$X \text{ or } S = \frac{a_2(R^n - 1)}{R - 1} = \frac{P(R - 1)(R^n - 1)}{R - 1} = C'\left[\left(1 + \frac{r}{q}\right)^{qs} - 1\right]$$

$$C' = \frac{X}{\left[\left(1 + \frac{r}{q}\right)^{qs} - 1\right]}.$$

Formula 22 is a very close approximation. To be exact, the numerator in the third term should contain $\frac{-X}{\left[\left(1 + \frac{r}{q}\right)^{qs} - 1\right]}$, assuming at the same time that n is a multiple of s.

As to the period of life of different portions of the water-works plant, opinions vary with the views and experience of the engineer or man rendering judgment, and many local circumstances and conditions affect the efficient or useful life of the structure and hence the depreciation very materially. Generally, however, the useful life of those structures has been assumed by different engineers, testifying in water-works valuation cases, to lie between the general limits given in Table 3.

TABLE 3.

	Useful life.	Annual Contribution to Depreciation Account or Sinking Fund in Per Cent. of Cost.	
		At 5% annual rate sinking fund.	General approximate results.
Reservoirs	50-100 years*	0.4777%-0.0383%	½-0%
Standpipes	25- 40 "	2.0952%-0.8278%	2 -1%
Masonry buildings	40- 50 "	0.8278%-0.4777%	1 - ½%
Wooden buildings	20- 50 "	3.0243%-0.4777%	3 1%
Cast-iron pipe of large diameter	50- 75 "	0.4777%-0.1322%	¾- ¼%
Cast-iron pipe of small diameter	20- 40 "	3.0243%-0.8278%	3 - ¾%
Steel pipe	25- 50 "	2.0952%-0.4777%	2 - ½%
Wood-stave pipe	20- 30 "	3.0243%-1.5051%	3 -2%
Wrought-iron service pipe	15- 30 "	4.6342%-1.5051%	5 -2%
Meters	20- 30 "	3.0243%-1.5051%	3 -2%
Hydrants	40- 50 "	0.8278%-0.4777%	1 - ½%
Gates	40- 50 "	0.8278%-0.4777%	1 - ½%
Pumping and auxiliary machinery	20- 30 "	3.0243%-1.5051%	4 -2%
Steam engines	15- 25 "	4.6342%-2.0952%	5 -3%
Boilers	12- 16 "	6.2825%-4.2270%	6 -4%
Electrical machinery	20- 30 "	3.0243%-1.5051%	4 -2%

Average for entire plant (gravity system) ..... 1% to ½%
" " " " (pumping system) ..... 2% to 1½%

* Except where subject to heavy deposit of silt.

The figures for depreciation in Table 3 may be largely exceeded. In a rapidly growing, young community, the efficient life of the structure may be very short; pumps may be outgrown and discarded within 5, instead of 20 to 30, years' time; pipe may be replaced in 10 years or even less, etc.; and yet reasonable prudence may have been exercised in the original design; but the promise of development, or direction of growth, or character of the demands of the service, may have been too uncertain to warrant larger outlay and more liberal design in the original construction.

In principle, therefore, every water company should "write off" or deduct annually from its assets a sum which will, at least, cover this depreciation, and the writer is of the opinion that, inasmuch as this allowance has to be made in advance of the development of the facts and testing of the structure, a liberal policy should be adopted in order to ensure reasonably the future integrity of the capital invested in the plant in so far as this phase of operation is concerned.

The total annual depreciation will generally be found to lie between the following limits:

For gravity works from ½% to 1%.
" pumping " " 1½% " 2%

of original cost, though considerable variations from these limits will be found, owing to the abnormally high functional depreciation resulting from errors in judgment, the development of structural weakness, faulty design, or from an over-niggardly policy in original construction, and to a multiplicity of other causes.

Sinking Fund as a Substitute for Depreciation.—As a matter of fact, while few municipalities have recognized this depreciation or perhaps even considered the underlying theory of it, they have, in effect, made allowance for it in the sinking-fund provisions of the bonded debt, and it may be added that the sinking-fund requirements have usually been in excess of the necessary allowance for depreciation, as a result of the assumption of a shorter life for the bonds than the actual life of the structures covered by them, though, perhaps, there is a tendency to-day to issue longer term bonds—the terms of which in some cases exceed the probable life of the structure. This practice is to be condemned unless the sinking-fund requirements correct this evil; even then this practice appears illogical.

Renewals.—Engineers are somewhat divided in opinion as to the necessity of charging off to depreciation any portion of renewals involving a betterment of the service. Some contend that renewals involving betterments of service—there being no difference of opinion, of course, in regard to renewals which are mere replacements of existing structures—should be charged in part only to construction; or, in other words, that the value in the original structure not previously written off in depreciation should be deducted from the construction account simultaneously with the addition to it of the increment of value in the renewal represented by the betterment of the service; while other engineers contend that the entire cost of the element of the renewal representing the betterment of the service, should be charged to construction account, without any deduction on account of the remaining value in the original structure which has been sacrificed or lost, upon the theory that the betterment is demanded by increased business or larger requirements, and that hence the new business should carry the entire burden of the larger debt thus created. To take a concrete example, let us suppose that the growing needs of a community necessitate the relaying of a 6-in. pipe with a 12-in. pipe. Upon the first theory the water-works company would be entitled to charge to construction account only the difference between the entire cost of the 12-in. pipe and the remaining value (*i. e.*, not yet written off) in the 6-in. pipe, the rest being charged to renewals in offset to depreciation; upon the second theory, the entire cost of the 12-in. pipe might be charged to construction account upon the theory that the 6-in. pipe was still adequate to meet the requirements for which it was laid, and that the larger main was required by the greater demands of added business which should therefore carry its entire cost. Furthermore, say exponents of the second theory, the saving in interest upon the smaller original structure during its life, as compared with the larger structure had it been built, has more than covered the cost of the smaller structure, to which it is replied, that this fact has no significance beyond pointing the limits of rational design.

Certainly the first theory is the more conservative and safer from the point of view of the bondholder or owner of the property.

Land, Water Rights, Etc.

Real estate must, of course, be valued on the basis of its market value and not upon the foregoing rules.

Monopoly Value of Land and Water Rights.—While the Maine Courts have laid down the rule that the public is entitled to the most advantageous development, it does not appear that consideration shall not be given to the fact that in some cases what may be loosely termed a monopoly value may attach to a property of this character. Take the case, for instance, of a reservoir site, where several alternate and equally advantageous sites are available, no particular value can attach to any one property in excess of its market value. If, on the other hand, there be but one site which can be used for this purpose without going to great expense in obtaining another, additional value in excess of the market value would appear to inhere in this property. Under some circumstances, perhaps, the limit of this value might be equivalent to a saving in construction upon this site, as compared with a less favorable site, or with the cost of an alternate method of accomplishing the same results, or with the additional cost of modifications in the design of the plant to meet the same ends. Generally speaking, however, this increment in value will be measured not in the manner stated, but by the additional sum of money which a purchaser would have been likely to give for this property when he purchased it, in advance of the construction of the works, in view of the possible use of this property by the company at a later date. A prospective purchaser would undoubtedly pay a larger sum for property with such inherent possibilities, but, on the other hand, if there was a possibility of his having to hold the land for a considerable period of years, he might consider this additional speculative value as small.

Commodity Value.

The idea of attempting to value water delivered through a pipe system as a commodity, appears to be fundamentally wrong except under such conditions as might prevail in a desert or where water is not to be found locally, or within reasonable distance for transmission by pipe system, and the writer has certainly not chanced to hear of any case in which valuation of water as a commodity has had standing. To state that water, where the supply of potable and suitable water is manifestly in excess of present or prospective needs of the community, can have value for many and diverse uses as a true commodity, appears contrary to fact. The value of the water to the community, aside from possible uses for the development of power, is created solely by the

demand for water supply for public, domestic and manufacturing uses in the particular community under question, and that, moreover, in a limited quantity. It has value only to the limit of the demand or market for it; in short, the only market for this water is a local one, and if there be several possible alternative sources of supply, the water actually in use derives its value, not from the fact that it is a commodity, but from the existence of the franchise under which the company is operating and from what might be otherwise considered thereunder as monopoly value in water supply, that is, the right to sell water in a given community for profit, usually under non-competitive conditions. The water has value to the company, not solely from the fact of its existence, but also from the company's charter or franchise right to distribute it and to collect toll; therefore, the commodity theory is weak in that it entirely overlooks the duty of the company, under common law rules and the Maine decisions, to furnish water service to the consumer at a price that is fair in any event. Far from being a commodity value, this value is predicated upon the fact of service and worth to the consumer, and is fundamentally dependent upon the franchise which gives monopolistic value to the water by prescribing non-competitive conditions.

Liability of Water Companies for Fire Loss Due to Inefficient Service.

A consideration which, heretofore, probably has had little weight in water-works valuation, but which may depreciate properties of this character permanently, is that of the financial liability of water companies for loss incurred by fire, in which inadequate fire service can be proven. Court decisions have been numerous upon both sides of this question, but the weight of decisions has—before the Greensboro decision—been largely in favor of the water companies, upon the technical ground that, inasmuch as the water consumers were not parties to the contract of the water company to furnish water to the city for fire protection (or, to use a safer word, water for fire service), they could not sue the water company for breach of contract. Judge Brewer of the Supreme Court in the appealed Greensboro Water Company decision,* however, has taken the view of water-company liability

*Guardian Trust & Deposit Company *vs.* Fisher, 200 U. S., p. 57, Brewer, 115; Federal Reporter, 184.

based upon the broad ground of negligence, a decision of far-reaching importance to many water companies, suggesting the need of the greatest care in the preparation and wording of fire-service contracts.

Going Concern Value.

The Maine Court has ruled that the "going concern" or "established business" is an element of structure value. It may be defined as the cost of establishing, up to the level of the net income of the old plant at the date of taking, the business of a similar new plant (but not of a more perfect system which the city might build), which is supposed to begin operation upon the date of the taking, the old plant going out of business at the same moment. Or, to put the same thing in another way, it is the measure of the greater value (at the date of taking) of an old, established plant, over a similar new plant completed and ready for operation upon the date of taking. Or, to put the same thing in still another way, it is the measure of the cost of developing the business of a new plant to a point coincident with the net income, upon the date of taking, of the old plant.

Hence the "going concern" or "established business" element of value—subject to the fairness of the rates and the character of the service—depends directly upon the net income or earning capacity of the works at the date of taking, and there does not appear to be justification for including in this value, as some experts in water-works valuation have done, consideration of the increment in net annual income in the period of years after the date of taking, during which the business is being established. The latter (increment) clearly pertains to franchise and not to going concern value. As previously stated, the going concern value is only one element of value, included in the total value found by capitalization or consideration of the net income under reasonable rates. To add the going concern value to the total value would involve double payment for the same thing, but in the case of the plant the franchise of which has expired, the going concern value must usually be calculated and added to the reproduction cost less depreciation thereon to date, in order to determine the value of the plant.

In passing it may be of interest to suggest certain underlying principles bearing upon the going concern element of value. In the course of their development and life history most water-works plants pass through the following phases:

1.—That of the plant which has not been operated for a sufficiently long period of years to develop its normal revenue or rate of growth, or, in other words, that has not passed through the formative period.

2.—That of the plant which has passed through the formative period, but which has not yet made good, through its increasing revenues, the losses sustained in the early years of the formative period.

3.—That of the plant in which the going concern has been established, and the losses during the formative period have been made good.

1.—If the works be taken by the city while they are still in the formative period, or period of business development, it is clear that the net revenue does not furnish an equitable means of determining the value of the property, since there is no net revenue, and this fact should be given due weight. If the owner of the works should attempt to sell his plant during this period, the purchaser or investor would probably give small recognition to this fact; the sale would be in effect in the nature of a forced sale, and hence the owner would probably not derive a reasonable return for his property, even from a willing purchaser. On the other hand, if the valuation of the works is based upon the taking by the municipality under right of eminent domain and hence upon their value to the owners, subject to limitation of reasonable rates and reasonably good and efficient service, as it must be under the Maine decisions, this fact must be given due consideration.

2.—If the works are bought immediately at the end of the formative period, but while the owner is still carrying the accumulated losses of the early years, this fact should be given due weight in arriving at a valuation based upon the earning capacity of the plant when taken by the city. The same idea might be stated differently by saying that, at such a time, the seller would not be a willing seller but a forced seller. It cannot be doubted that if the works should be purchased by an investor or a willing buyer at that time, he would make no allowance to the owner for any losses incurred by him in the early years of operation, except in so far as prospective earnings might warrant some small consideration, for the value would be derived and

the market price be paid as a result of the going concern, the established business, and the earning capacity, and not as a result or measure of the works having passed the early hazard period. In the case of a plant taken under the right of eminent domain or legislative authority, however, the valuation must be made on a basis of the value of the works to the owner subject to the limitations of reasonable rates and service, in which case the fact cited above should be given due weight, and, in case of fair-rates suit, due consideration should be given, in a study of the rates, to the fact that the owner had not been reimbursed up to that time for the early losses, and this condition would result in the postponement of the day when the community might justly expect a reduction in the rates for water service.

3.—Finally, in the case in which it is assumed that the company has been reimbursed, through the agency of the rates, for the losses during the early years of development, the full value of the going concern has been established, and the owner has received adequate return upon his property up to date of taking; and it may be added that a new situation has arisen, under which the rates, previously equitable, are beginning to be unjust to the community, unless increased extensions and improvements to the property or service make a drain upon the revenue commensurate with the surplus earnings which had previously been applied to the liquidation of the early hazard debt, or the losses incurred in establishing the business.

If it is a fact that after a reasonable period of growth, which period could not well exceed ten years, and, in many cases, a less period, the normal income of the company has not been developed, and it is still being operated at a loss, no other conclusion can be drawn than that the owner of the property has voluntarily taken upon his shoulders a losing contract with the community, the penalty of which he must, under the terms of that contract, pay, without increase in rates and with disadvantage to himself, in case he should attempt to sell the property. As owner he was not obliged to enter into contract with the community to supply water to it, or to accept the franchise which it offered, but having done so, under the law of contracts, he must suffer the consequences to the limit of his financial ability. These losses, under the circumstances cited, would be the measure of the price which the owner would have to pay for the privilege of the continued enjoyment of his franchise.

In the writer's experience, the going concern value has usually been found to be between the net and the gross income of the plant for a period of one year (at the date of taking). It may be largely affected, however, by the period required for the development of the business.

Franchise Value.

The franchise rights usually cover fundamentally the right to take and develop sources of water supply, condemn property necessary for the use of the water-works, use the streets, highways and public places, supply water to consumers for domestic and commercial purposes, and to collect tolls or water rates therefor.

Its value cannot generally be determined directly. If desired, it must be obtained by eliminating from the total value of the property the remaining elements of value, if this is possible.

The Measure of Fair Rates.

Much has been said by counsel in different cases upon the unreasonableness of the rates, with but little effect, however, as no measure has been given of the extent or degree of the unreasonableness or of its effect upon the consumers individually or collectively.

Fair rates cannot be determined by comparison of water rates in neighboring or similar communities, since identical conditions probably never exist, and weight cannot be given readily or intelligently to the different causes which might affect them. The Court has always recognized this fact and wisely excluded direct comparison of rates. Nevertheless, it is probably true that rates in similar communities do have weight in limiting the magnitude of the rates.

It has remained for Judge Savage to furnish the proper gauge or yard-stick with which to test the fairness of the rates—the worth of the service to the consumer, as well as the fair return to the water company.*

Here is a frank recognition of the obligations of the public servant on the one hand, and of its rights on the other. The water-works corporation is not engaged in a private business for profit, limited only by competition and the laws of supply and demand, but in the performance of a public duty, subject to the requirements of efficient service and fair rates and privileged to make profit under non-competitive condi-

* See 99 Maine, 371, Pars. 6, 7, 14 and 13.

tions. The property of this quasi-municipal corporation, therefore, cannot have, or be valued upon the basis of, "market value." In appraisal, its value, under the Maine opinions, must be the "value to the owner," subject, however, to the limitations of the efficiency and the worth of the service to the consumer, or, in other words, of fair rates for water service of the character furnished.

Obviously, it cannot be expected that the cost of this service will not exceed the cost to the public of supplying itself; for it is reasonable to suppose that, inasmuch as the public elected for one reason or another to grant the franchise for water-works to a corporation, it was willing to pay somewhat higher rates than those at which it might have supplied itself. This excess is a matter of judgment in the light of all the existing facts, but the Court calls attention to the need for caution in applying this rule, since the company, in accepting the franchise, voluntarily chose to take whatever hazard there was in the business; and further holds that whatever the consequences or loss to the owner, the rates must not, in any event, exceed the worth of the service to the consumer.

Heath's Method of Measuring Fair Rates and Value.—As counsel aptly put it, one cannot value without rating, and once one has determined the fair rates for a given plant, one has valued it.

The method of rating and valuation outlined herein was first developed and used, so far as the writer knows, by Hon. Herbert M. Heath, of Augusta, Maine, in the suit of the Kittery (Maine) Water District *vs.* the Agamenticus Water Company, in November, 1907, and later in two other water-works valuation suits—Livermore Falls Water District *vs.* Livermore Falls Water Company, heard in November, 1907, and the Portland (Maine) Water District *vs.* the Portland Water Company, the Standish Water and Construction Company, the Gorham Water Company and the Foreside Water Company, heard in February to May, 1908. Mr. Heath had previously had considerable experience in water-works litigation and valuation, and had given much study in his wide practice, as a corporation lawyer in New England, to the important economic questions involved in the relations of the quasi-municipal or public service corporations to the public; but it was not until the Brunswick decision that this principle of the worth of the service to the consumer was laid down by the Court, and first put to practical application by Mr. Heath. As the writer had the pleasure

of assisting Mr. Heath in working up his theory (or method of applying this principle) and the tabulations for his argument in all three of the above-mentioned cases (Kittery, Livermore Falls, and Portland), he may be permitted, perhaps, to outline it.

The theory was probably the outcome of a critical study, by Mr. Heath, of the rating question, of the rules of valuation and fair rates laid down by Judge Savage in the Waterville and the Brunswick opinions, of the previous valuation litigation in Maine and elsewhere, of the English methods which governed the valuation of the London water companies, and of his interest in the broad economic principles and questions of public policy involved in the relations of quasi-municipal corporations to the public.

Value, Mr. Heath holds, must be based upon cost—be the latter reproduction cost, based upon prices prevailing on or about the date of taking; or reproduction cost based upon fair average prices for a considerable period of years; or original cost; or something between these three—upon cost plus a profit which shall be limited, upon the one hand, by what may be needed to induce investment in and active competition for the privilege of operating public service corporations, and, upon the other, by the cost at which the public might serve itself equally efficiently plus the profit which the public might be willing to allow to a corporation for doing the work and thus relieving it of the necessity and, perhaps, risk of serving itself.

Starting with this fundamental idea, that the public service corporation is entitled not only to the reasonable cost of furnishing the service, but also to a reasonable profit thereon (assuming always that the works can earn the necessary income to admit of this), we come logically to a consideration of the principal elements or factors to be covered by the rates, and hence to what might be termed the maximum lawful income for the given plant under existing circumstances—for any increase in earnings beyond this point should, by the law of diminishing returns, revert or inure, sooner or later, to the public, in one form or another. To say that, in a consideration of the lawful annual income to be derived from the rates, no allowance, beyond operating expenses, depreciation, and a reasonable return upon invested capital, should be made to the builder or owner of the works, would be contrary to public policy and expediency. Under such conditions capital could not be induced to embark in such ventures, and progress might be seriously handicapped in many communities.

Elements to be Covered by Lawful Rates.—Subject to the limitation of the Court ruling, that the rates shall be reasonable to the individual as well as to the community as a whole, and to the practical business possibilities involved in the case or the willingness of the people to become water takers at the established rates, the corporation is entitled to fix its rates so as to earn:

First.—Full operating expenses, including therein all necessary repairs to keep the plant in good condition.

Second.—A depreciation or renewal fund, which shall always keep intact the capital invested, by replacement through this fund of the worn out or described parts of the plant.

Third.—Such a rate of interest upon the invested capital as will correspond to the degree of hazard involved in the investment.

Fourth.—Interest upon the reasonable profit increment (discussed in the following pages) which is assumed as compensation for undertaking this public service.

Fifth.—A sinking fund, if approved by the public. The question as to whether a sinking fund should or should not also be earned and allowed is perhaps a "local-option" question, inasmuch as it merely affects the distribution of the burden of construction, and not its amount.

If a sinking fund be added, the item of interest upon invested capital included above would be omitted, of course, after the amortization of the debt of construction by the sinking fund. If the annual contribution to the sinking fund be large, the construction debt will be quickly paid, but the present burden will be heavy upon the consumer; if it is small, the period will be lengthened and the burden decreased. Views differ upon this question which is after all one of sound public policy. The occasional needs or burdens growing out of war, fire, earthquake, flood, or other disaster, which have befallen cities from time to time, dictate or at least indicate the wisdom of letting each generation bear its own burdens and of making regular contribution to a sinking fund, which will wipe out the construction debt within a reasonable period of time, such as 20 to 40 years.

Referring now in greater detail to these elemental considerations in determining rates, the "operating expenses" need no comment, and

"depreciation" has already been discussed;* the "rate of interest," the "base upon which it is to be figured," and the "reasonable profit" are not so easy of determination.

The Rate of Interest.—This rate should not be that at which the city or town could borrow money, even upon its water-works securities, as the community's credit rather than the actual hazard would be reflected in this rate; neither should the rate at which the company has borrowed money be controlling, though it may be confirmatory evidence. The customary rate of interest prevailing upon securities or loans of equal security or hazard should govern. In New England, rates of 5 to 6% may be considered, perhaps, as typical of average conditions in recent years.

Under the method of valuation discussed herein, the rate of interest actually adopted and used makes no difference in the final valuation of a plant which is earning its maximum lawful return. It pertains only to the rating, to the determination of the maximum lawful income from the given investment under existing conditions. As far as the value goes, it makes no difference whether the rate be assumed to be 5 or 10%, for having once determined the lawful, annual income (based upon this given rate), the value may be obtained by capitalizing this income at the assumed rate of interest, or, what is exactly the same thing, having determined the base on which interest and profit are to be calculated (in the building up of the lawful income), the value is obtained by increasing this base by the profit increment, or ratio, to be allowed for assuming this public duty or performing this public service. This subject is further discussed under a later paragraph upon "Reasonable Profit."†

The Base on Which the Interest is to be Calculated.—This factor requires the most careful thought. Should interest be computed, first, upon the reproduction cost at the time under consideration; or second, upon a fair average reproduction cost during a period of years long enough to give normal or mean business conditions; or third, upon actual cost of the works? The Courts have prescribed the first, as one of the fundamental considerations of value, but they have also attached great value to the last. The second method has always seemed to the writer, on the whole, the most equitable of the three.

* See pages 1116 to 1125.
† See page 1136.

In figuring the cost to the public of supplying its own service, this cost may be based upon the reproduction cost of the plant, upon the theory that the rating is made as of the date of taking, and current prices, therefore, should control, or upon the original cost, on the theory that the company is entitled to a consideration of its actual investment. Each method has its objections, the former, in that in a period of exceedingly high, if not actually inflated, prices for materials and labor, such as that through which we have passed recently, the reproduction cost would be abnormally high and as injurious and unfair to the public as would be the conditions to the company under abnormally low prices; the latter, in that the town is no more entitled to the benefit of the particular advantages accruing to the company for unusual good fortune, or economy of or shrewdness in construction, than it is to be held responsible for any negligence, bad judgment, or lack of prudence, foresight or economy on the part of the company. There would seem to be some justice, therefore, in basing the cost-to-the-public-of-furnishing-the-service (as a gauge of fair rates to the company) rather upon what might be termed the fair average prices for a period of years, than upon the actual reproduction or original costs, though the Maine Court has said that reproduction cost should be figured on actual prices and that the owner is entitled to any appreciation, or must suffer the loss due to any depreciation, growing out of change in prices since the original construction.

In the case of a limited and expiring franchise, too, the reproduction cost, as of that date, decreased by the depreciation to date and increased by the going concern value, would seem to furnish the proper basis for valuation under existing laws, though, in equity, it would appear fairer to compute its reproduction cost on average prices prevailing over a past period of years, rather than upon actual prices at the time under question.

The Reasonable Profit Increment or Allowance.—The reasonable ratio or excess of profit over cost to be allowed to the public service or quasi-municipal corporation is largely a question of good sound judgment based on an intimate knowledge of all the facts and the local circumstances bearing upon the case in question. Generally speaking, this ultimate ratio, corresponding to the development of the maximum lawful income for any given investment, may lie between 25 and 50%, but even these limits may be exceeded.

With good, energetic and thrifty management, the maximum development and profit increment will be reached early in the history of the plant and may be maintained throughout its life; with bad management, its development will be delayed or may never be reached, and even the original investment or capital may be impaired or lost.

The profit ratio, as indeed the interest rate, is thus dependent on the character and stage of development of the plant; on the magnitude of the project; the character of the community, its size, rate and permanence of growth, wealth, comparative safety, stability, prosperity, liberality, nationality; on the attitude of the community and of the courts; and on the character of franchise; physical environment as affecting possible competition; the character and efficiency of the physical plant; the physical risks; the provision for future needs, etc.

In this connection attention should be called to three considerations, sometimes lost sight of, which should be borne in mind in reviewing past operations of any corporation with reference to present valuation or rating:

First.—The difference in the water-works standards of to-day, as compared with those of the past.

Second.—The change in attitude of the public toward quasi-municipal corporations, the progressively closer public supervision of their operations, and the greater limitations put upon them.

Third.—The extent to which the past operations and earnings of the company, at rates which have not been challenged for a long period of years, may be presumptive evidence of their supposed fairness from the public point of view regardless of actual cost to the company of furnishing the service, and even though they may show to the company an unreasonably large profit upon the actual capital invested.

Fourth.—The effect of past determination of value or of fair rates by legal or judicial procedure after agitation of these questions.

The third consideration is one of the most difficult to handle, for, on one hand, innocent investors who have purchased water company securities upon a basis of their earning capacity or market value,

under rates long in force and never called into question, are ranged against the public with its right to demand efficient service at rates that are not exorbitant to the water consumers, either individually or collectively, and that shall show to the company and its investors no more than a reasonable profit upon its property.

If the property or rates have been subject to controversy and legal adjudication prior to the valuation, the results reached at that time would seem to furnish a suitable starting point for subsequent valuation, and, perhaps, prevent the possible oversight of the elements referred to.

Disposition of Excessive Rates.—If, judged by the worth of the service to the consumer, the rates are excessive, the excess should apply to reduction in rates, betterment of service if the latter be needed, or to reduction in debt; and any increase in net annual income (over and above the demands of increased operating, maintenance, or fixed charges growing out of additional investment) should apply to the same end—reduction of rates; or, as Mr. Heath, Counsel for the Livermore Falls Water District, aptly put it, in his final argument:

"* * * Any additional net income derivable here would be excessive; and we would approach that stage in rating when, as they call it in the books, under the law of increasing returns, the new additional net income should inure to the benefit of the public served, * * *"

and Mr. Heath went on further to draw a parallel with the history of the railroads of the United States which, with increasing tonnage, have been able to reduce their rates enormously.

"This is but the natural operation of the economic law in public channels, a recognition of it by men who knew and understood that they were public servants; and that as the business increased the public got the benefit of it.

"* * * We have shown that the day has come in the history of this plant when the future increment in income belongs to the people by way of reduction of rates, a just principle, Mr. Chairman, because a certain amount having been invested by somebody, having built a good plant, having covered the area, then there should be an hour some time in the history of this and all water companies when they reach the maximum net income that is lawfully derivable, holdable and keepable from that plant; and when you go beyond that, the hour of the people has come to receive proper reductions in their rates. Now assuming that I am right in these propositions, let us view the matter from the standpoint of the element of fairness to the company. The Court has said * * * in this Brunswick decision, * * * that

the amount of money actually and wisely expended is a primary consideration in this matter of a fair return; and it ought to be, because the money invested ought to be protected by the courts and legislature, so far as can be done with justice to the public; and if it be that they can get a fair return upon money originally expended, at the same time doing it without oppressing the people, and not exceeding the worth of the service to the community, they ought to have it—otherwise we would have no water-works."

The owner of the water-works benefits not only by the allowance to him of a return in excess of the cost to the community of furnishing a like service, but also by the greater security of his investment growing out of the just recognition of these principles.

Extracts from Argument of Hon. Herbert M. Heath, Counsel for Water District in Portland, Maine, Case, Upon Method of Valuation, Reasonable Profit Ratio, and Rate of Interest to be Allowed Upon Capital Used.

Mr. Heath outlined the foregoing method of valuation so clearly in his argument for the Portland Water District, May, 1908, that a portion of his argument is here quoted:

Extract from Outline of Method of Valuation.

* * * * * *

"The method therefore assumes the appreciation of the valuation rules of the Waterville and Brunswick cases to this plant and its franchises, as now developed, the reproduction cost standing as the equivalent of the capital employed, earning the maximum profit lawfully derivable from it, with a charter giving the right to earn from takers the full profit ordinarily derivable from all its service, with its present lawful profit assured, with no depreciation, the quality of the service unattackable, the plant now fit to meet reasonable requirements of the present and future, efficient in every part, having kept pace with and reasonably anticipated municipal growth, with no acts of its own permanently depreciating its franchises, with no property not reasonably necessary to the service, with no structure unreasonably expensive, and, finally, with an assumption that the works were built prudently.

"A valuation so reached treats the property by assigning to it full value for all its characteristics of value, as a structure, as a structure that is a going concern, and as a structure with a right to continue the use of its franchises, that is, to earn the maximum profit derivable from it by law without new development. The result must necessarily

be the largest possible valuation assignable to the property and franchises, due allowance, if any, to be made later for new development.

"After reaching such a result, we shall then contend that it ought to be reduced because of the following depreciating elements:

"1. That the customers of the company cannot lawfully be made to bear any part of the burden of the worth of the free municipal service.

"2. That the plant has suffered in value from physical depreciation.

"3. That the quality of the service rendered is below the reasonable standard required by the charter.

"4. That the plant is not now fit to meet reasonable requirements in the present and future, is inefficient in whole and in parts, and that it has not kept pace with or reasonably anticipated municipal growth.

"5. That by its transactions with the Standish Water and Construction Company, and the distribution of the stock received for the perpetual water supply, the Portland Water Company has seriously and permanently lessened the normal enhancing power of its franchise characteristic of value, and thereby materially and heavily diminished the value of its property and franchises.

"6. That its transactions with the Foreside Water Company have likewise permanently diminished the value of its property and franchises.

"7. That the lease of the Sebago basin was not reasonably necessary to the service.

"8. That various structures were unreasonably expensive and were not built prudently.

"9. That, finally, due consideration must be had of the original cost of the plant in use, and the final result reduced as fairness to the company and to the public may require.

"Thereafterward, study must be had of the value, if any, of the right of the Company to extend the use of its franchises by new investment and, if the facts require it, addition be made therefor.

"Next, should be discussed and properly treated the claims of the Company for additional value for the Bramhall and Munjoy Sites, the services, the public hydrants, and the Warren Contracts. All these elements we shall claim to be without value.

"Then to reach the award must be added the agreed value of the tools and supplies, the value of the Martyr Street Lot, the allowance (if any) for maps and plans, and the values of the stocks of the Gorham Water Company, the Foreside Water Company, the Presumpscot Water Power Company, the Saccarappa Aqueduct Company and the Munjoy Land Company.

"The result will be the award to the Portland Water Company, as we claim it ought to be.

"We wish, at the start, to emphasize the statement that any result reached in this case by starting with the reproduction cost as a basic factor must necessarily be too high. The prices of the reconstruction period involved in the conception of building this plant for delivery on January 1, 1908, were abnormally high.

"Care was taken in the Waterville case, p. 207, to say that caution must be used if it appeared that the works were originally built when prevailing prices were high so that actual cost might exceed present value. The suggestion is equally forceful in using the element of replacement cost computed as of a period of abnormally high prices. And the Court took great pains to emphasize the importance of original cost in its relation not only to present value but equally so in the determination of the maximum profit legally derivable from the property.

"Great care was also used throughout both the Waterville and Brunswick cases to say that, in arriving at present value, neither original cost nor reproduction cost should outweigh the other, but that both elements should be carefully considered.

"The practicable method seems to me to estimate the award first by using the reproduction cost as the basic factor and next to so estimate it by using the original cost, leaving it to the Court to work out the truth somewhere between the two, as justice may require.

"The method proposed ultimately appraises the system in use, in its present condition and with its present efficiency. (Waterville case, p. 216.)

"It rejects the capitalization of profits, condemned as improper in the Waterville case.

"It does not undertake to separate the award into structure-value, going concern value and franchise value, condemned in the Brunswick case as inherently impossible.

"A method that completely values property and franchises as a unit values the property as a structure, as a structure in use as a going concern, and as a structure which may rightfully continue to be used as a going concern structure. (Brunswick, p. 376.) The owner thereby receives full and fair value and the just compensation required by law.

"Throughout the discussion it must be remembered that while the present depreciation is heavy, due to physical wear and tear and to inefficiency calling for new investment to an amount so large as to seriously lessen present value, the Company has had net profits enough during the past forty years to have given to it a full fair return from the very first year and to have also accumulated from its remaining profits a depreciation fund large enough to meet every dollar of the depreciation now due to deterioration and present inefficiency due to the fact that the territory served has outgrown the capacity of the plant as it has three times before in its history.

"Of this accumulated depreciation fund the Company now has on hand cash assets of the value of nearly $300 000, all of which it retains under the taking act. It has earned rates in the past upon the theory that this fund was available to make good all depreciation due to deterioration of plant or to loss in value of any part of its system due to growth of the public needs. The fund on hand is the lawful and legitimate accumulation of the depreciation allowances with which it properly loaded its rates. If it is to keep it, the Company cannot complain that the value of its plant should be lessened, in the award, to the extent of the depreciation, whatever it may be.

"In addition to this $300 000 (approximately stated) the Company has, as above stated, also collected from its takers in the past in addition to a full fair return from Jan. 1, 1870, enough to meet every dollar of present depreciation, physical, functional or otherwise. This we will show from the books before the case closes. Instead of keeping it to meet death of capital in the public service it has divided it. It has been dividing its plant for twenty years, as do so many corporations greedy for dividends on stock that represents no investment. We shall show later that this Company has received in rates enough so that if it had been content with drawing as dividends simply the profit allowed by law, it would have had on hand to-day every dollar that we now claim should be deducted for depreciation or loss in value of every kind and nature. It has made its own bed and is the author of its own misfortunes. To refuse to deduct the depreciation from present value is to pay this corporation twice, once in the award and once in the depreciation charges with which it lawfully and properly loaded its past water rates.

"Every public service corporation has the right to load its rates with an item for depreciation, present and prospective, that will prevent any possible diminution of capital value from use in the public service, whether wear and tear physically or from loss of efficiency due to the fact that the territory served outgrows the capacity of the main features of its system. The Portland Water Company had that right. It exercised it. It collected the amount. It would to-day have been its full protection. It need not have lost a dollar in this award because of depreciation of any kind. It divided a part of the money and has the rest on hand. It has no one to blame for its folly but the greed of its stockholders and their want of business foresight. Having seen the city three times outgrow the capacity of the system, they should long since have prepared themselves for the emergency that faced them long before the Water District agitation began.

"Equally so of the serious loss in value due to the distribution of the Standish stock. When the stockholders of ten years ago lugged off the Standish stock, and left to the stockholders of to-day the burden of forever supplying the Standish Company with water without pay,

they should have foreseen the present necessary and inevitable financial consequences. That the Portland Water Company of to-day must suffer heavily in value because of the greed of its stockholders of ten years ago is plain. The corporation has no one to blame for its present misfortune but itself. Five-sixths of the value of the Standish equity ought, in all equity and good conscience, to go to the Company that to-day and forever must carry the burden of the supply works needed to furnish the Standish Company with water. It goes to strangers. This is not the time to argue the point. I throw out the suggestion now simply because here, as well as in the matter of the depreciation losses, the loss in present value is the necessary consequence of the avarice and greed of the stockholders of the Company.

"The award should be fair to the Company, but it should be paid just compensation but once."

* * * * * *

Extract from Maximum Valuation Assignable to Property and Franchises.

"Continuing the assumption that the cost of reproduction represents the equivalent of the capital employed, the plant earning the maximum profit legally derivable from it, with a charter giving the right to earn from takers the full profit ordinarily derivable from all the service, with its present lawful income assured, with no depreciation, the quality of the service unattackable, the plant now fit to meet reasonable requirements in the present and future, efficient in every part, having kept pace with and reasonably anticipated municipal growth, with no acts of its own permanently depreciating its franchises, with no property not reasonably necessary to the service, with no structure unreasonably expensive, with the works prudently built, what is the largest possible valuation that, with no new development, can be assigned to the property and franchises under the valuation rules of the Waterville and Brunswick cases?"

Extract from Ratio of Profit on Cost of Capital Employed.

* * * * * *

"We recur to the original proposition that whatever percentage or ratio of profit is selected to increase the return on capital employed beyond its annual cost or worth, necessarily just so much increases the value of the capital employed. And, inasmuch as this increment is due wholly to the maximum use of the franchises, the capital employed increased by the percentage selected as the ratio of profit gives the largest possible value assignable to the property and franchises (without new development), all premised upon the assumption that there are no depreciating factors. Deducting from this result the proper allowances for depreciating factors, whatever they may be, and we

have the present value of the property and franchises as they were in fact on January 1, 1908.

"It is not an easy matter to determine just what this ratio of profit ought to be. Upon its correct and fair determination rests the justice of this award, both to the public servant and to the public served.

"Were this a rating decree, this ratio of profit would determine the maximum rates to be paid. In that event the basic rate of interest to be allowed for cost of capital employed would be highly material.

"It is plain, as a simple principle of mathematics, that it is immaterial in a valuation decree what rate of interest is selected as the cost of capital, provided it be shown, as will be shown later, that, testing the rates, as charged, by any basic rate of interest and any ratio of profit that this Court would be likely to approve, the present return is legally excessive.

"The only law upon the point is the remark of the Court in the Brunswick case (p. 386) that the community may with reason prefer to pay rates which yield a return to the money of other people, higher than the event shows they could serve themselves for, rather than make the venture themselves and risk their own money to loss in an uncertain venture."

* * * * * *

"It may not be without value to make some study of public service corporations and their relations with capital.

"The data do not exist for studying the ratio of profit in other water companies in this State or elsewhere. No book is published giving the statistics of water companies. Water companies have hitherto been able to defeat all legislation tending to publicity by way of annual returns. Even if made, the usual practice of falsifying the cost of construction, as in this case, would make the returns useless as evidence.

"A study of municipal water plants is futile. Their ratio of profit is often a minus quantity met by taxation. When a profit over cost is found, it is usually because the municipal policy is making the ratepayers pay for extensions out of earnings, or hastening the payment of bonds, or some other consideration to make the people believe that they are acquiring property without being taxed in fact.

"The only exhaustive and educating returns are those of railroads. They approach the standard of the water company nearer than any other public service corporation.

"The street railroad is yet too crude. Mechanically it is in its infancy. Its depreciation factor is as yet an unknown factor. Its fares are largely arbitrary and a matter of custom, not established in any sense on scientific principles.

"The electric light has, as yet, too many unsolved problems of maintenance and heavy unknown depreciation. Equally so, the telephone.

"The steam railroad, by long use and experience, by its closeness to the public, by partial public regulation, and by a growing system of scientific official returns, is somewhat analogous to the water company.

"Still the analogy is not perfect. The business risks of a railroad are great. The opportunities for loss through negligence constantly threaten earnings. There are many peculiar hazards and difficulties. It is dependent upon the general prosperity of the country in a large degree. Competition is usually a factor. It is far more subject to restrictive legislation. In short, it by no means presents the safety and security incident to an investment in a water plant in a prosperous city. And yet it approaches the safety and return and security of investment in water-works more nearly than any other public service corporation.

"A long and patient study of the official returns of the Maine Central Railroad covering several years, leads me to the conclusion that, applying to them the reasoning here used, its ratio of profit to capital cost is 1.26, and the Boston and Maine, 1.24. The Somerset Railway, an old road, 1.28, and the Bangor and Aroostook, a newer road carrying many risks, 1.40.*

"The expert for the Census Bureau, Professor Henry C. Adams, Statistician to the Interstate Commerce Commission, in 1904, computed the commercial value of the railways of the United States at $11 244 852 000. On this basis their full net return in 1906 was $6\frac{1}{10}$ per cent. The statistics for 1907 are not yet made up.

"We respectfully submit that it is more than just to a water company, situated as this Company is, to fix its ratio of profit at 1.25. Assuming the cost of capital to be $5\frac{1}{2}$ per cent., the maximum rate would yield $6\frac{7}{8}$ per cent. on the capital employed as represented by the cost of reproduction." * * *

"We submit that capital will gladly enter the water-works field, whenever it can be assured by law that the full use of the franchises will enable it to receive from the public compensation for all original risks, full compensation for all capital used up in the service, an annual return upon the value of the property and franchises at the going rate of capital invested with like degree of safety, and that it may step out of the enterprise with such natural increment as may come to it, and with a profit equal to one-fourth of the original capital and its natural increment. It has earned in the meantime not only a return on the capital itself, measured by the going cost of such capital, but a like return on the natural and franchise increment. Such an opportunity and such remuneration is attractive to capital anywhere."

* * * * * *

* Mr. Heath really means the ratio of the cost plus profit increment to the cost is 1.26 to 1, etc., or, in other words, the profit increment of the Maine Central Railroad is 26%, Boston and Maine Railroad, 24%, Somerset Railroad, 28%, and Bangor and Aroostook Railroad, 40%.

"Renewing all these considerations, the fact that original risks have long since been paid for by the customers, that the days of hazard and difficulty were long since passed and paid for by customers, that the Company has once lawfully collected from its customers through depreciation allowances in the rates enough to make good all present depreciation, whether physical or functional, that the investment is now as safe, stable and assured as any form of private investment, that its prosperity is independent of panics and hard times, that the business is a practical monopoly, guarded by Courts and the Constitution of the United States, situate in a city with stable prosperity and serving a people respecters of equity and of law, with a system mechanically simple and comparatively free from loss through accident or negligence, independent in its earning power of the political hysteria of city government, in short, surrounded with the best attributes of safety and security, we submit that the ratio of profit of 1.25 is fair to the Company and that to allow more would be unfair to the public. * * *"

Application of Foregoing Principles in the Livermore Falls Water-Works Valuation.

As an illustration of the application of the foregoing principles, the Livermore Falls case has been selected from the three recent Maine cases cited, not on account of its magnitude, for the amount involved was small, but because it lends itself particularly well to the method of treatment used, and affords a good illustration, as all the essential facts were well established in the evidence submitted.

Data Relating to Community.—Town on Androscoggin River. Location due to fall in river and water-power development there. Power owned substantially by one corporation, International Paper Company, and all used except further down stream. A one-industry town, and that industry one which requires comparatively few employees. Considerable foreign population (mill hands), many living in frame tenement houses. Population less than 3 200, by count. Census statistics not applicable, on account of boundary lines between towns. Future growth doubtful, probably not large. Assessed valuation (figures not in the writer's hands at present).

Legal.—Date of franchise, 1889. Perpetual franchise. Exemption from town taxation for ten years, expiring a few months after date of taking. Works built in 1899. Date of taking, October 1st, 1907.

Physical Plant.—Gravity supply—maximum head approximately 125 ft., minimum 0 ft. Town on hillside, business district at lowest

level, churches and many houses on a sort of middle bench, remaining residences on hillside above, many of them unable to get water on account of lack of available head or pressure upon the pipe system.

Source of Water Supply.—Moose Hill Pond. Drainage area 0.672 sq. mile. Water surface 0.142 sq. mile = 21.8 per cent. Storage per foot depth, 30 000 000 gal. = 45 000 000 gal. per sq. mile of drainage area (for depth of 3 ft., 125 000 000 gal., and 5-ft. depth, 200 000 000 gal. per sq. mile of drainage area). Quality admitted excellent. Quantity, safe daily yield 450 000 gal. (by agreement of engineers). Water consumption approximately 300 000 gal. per day, or 95 gal. per capita. 335 taps; 520± takers, substantially all on flat rate. No high-service system, though contemplated by company for several years, and legislative authority was obtained to issue securities therefor some years before taking.

No reservoir other than pond, 4 miles from town.

Pipe Distribution System.—Cast iron, chiefly. Ample in size for the most part, and generally efficient except for fire service. Service adequate for probable future needs, except for elimination of some small pipe and construction of high-service system.

Fire Service.—In business district, two "fair, ordinary fire streams" 185 gal. per min. 44 ft. high, 44 ft. horizontally. Middle bench or portion of town, "two feeble fire streams" 168 gal. per min. 36 ft. high, 38 ft. horizontal. Upper levels, no fire protection except from one fire steamer owned and maintained by town.

Operating Statistics. Financial Data.—

Gross Income, Operation and Maintenance and Net Income.

Year.	Income from:			Operating Expenses.	
	Water.	Service pipe, etc.	Total.	Exclusive of interest and depreciation.	Net income.
1900*	$2 203.88	$49.86	$2 253.74	$342.42	$1 911.32
1901	3 359.13	129.15	3 488.28	344.01	3 144.27
1902	3 447.75	507.60	3 955.35	401.52	3 553.83
1903	4 246.26	216.81	4 463.07	661.86	3 801.21
1904	5 163.03	69.26	5 232.29	393.96	4 838.33
1905	7 020.88	157.73	7 178.61	733.71	6 444.90
1906	7 524.80	174.17	7 698.97	801.63	6 897.34
1907†	6 862.63	71.72	6 934.35	739.78	6 194.57

* Thirteen months from November 29th, 1899, to December 31st, 1900.
† Nine months.

The estimated rate of annual gross income as of date of taking was $9 364.50.

The operating expenses are very misleading, since no town taxes were paid by the company (under a ten-year exemption which was about to expire), and as the operators of the works were willing to forego salaries in lieu of dividends. The estimated reasonable annual operating expenses for current year, agreed to by the engineers for both parties, were:

Reasonable Annual Operating Expenses, Current Year.—

Wages (including administration)..... (Charged by Company, $176.65.)	$600.00	
Repairs, as charged by Company.....	164.52	
Expenses, average since 1901.....	245.07	
Taxes (Town tax, real estate, state and county, on plant).....	290.10	
Town tax, 17 mills on, say, $60 000.....	1 020.00	
		$2 319.69
Annual depreciation (Metcalf).....		450.00*
Total.....		2 769.69

Depreciation (annual) $450.00 (Water Company's engineer estimated $345).

Resulting net annual income exclusive of interest charges $6 594.81, say, $6 600.

Unit figures. Net earnings (including hydrant rentals but exclusive of depreciation and interest and dividends)—

$60.00 per million gallons.
19.70 " tap.
12.70 " taker.
2.06 " capita.

Average Annual Return to Builders of Plant.—

Amount paid in by stockholders, $1 000.

Dividends (1899-1907).....	$10 000.00
Surplus on hand.....	3 965.36
Excess put in land chiefly for sale of lots and virtually a land speculation.....	11 700.00
Street work (grading) upon same.....	4 229.00
	$29 894.36
(Average per year, $3 857.)	
If to this be added the interest paid to bondholders..	25 320.41
	$55 214.77†

*Sawyer, $258.90 + $86.30 = $345.20.
†Of this amount, however, $3300 should be deducted as representing salaries not drawn.

Average per year		7 124.69
Percentage per year on investment of $65 477.61 plus $1 000		10.7%
Real estate purchase price (October, 1901)		$4 000.00
Estimate of market value by Company's officers		7 800.00
Estimate of market value by Town's witnesses		1 800.00
Structure value. Original cost by Company's books (admittedly approximate)		65 477.61
Reproduction cost by agreement of engineers, service pipes excluded, claimed by Company, but not admitted by town, as they were laid at cost of the consumer. Value, $1 845. Tapping fees collected, $1 806. (Both figures agreed to by engineers, subject to inclusion or exclusion by tribunal)		85 394.36
Supplies on hand		1 109.90
Depreciation.—By Company's expert—Annual	$345	
" Town's " "	450 = 0.527 of 1%	
Total to date = $1 800 ?		
" " " = 3 455 = 4.05%		
Cost of High Service.—Admitted necessary by Company; estimated only by Town's engineer, but not contested by Company as to fairness of estimate		$17 300.00
Additional annual expense involved in operation, maintenance and depreciation of high service	$600.00	
5% interest upon additional investment (no additional taxes figured here)	865.00	
		1 465.00
Probable increase in revenue (estimated by Town's engineer)		986.00
Present net annual loss		479.00
Probable loss in value on this account (up to time when new business would carry the improvements)		8 000.00
Going Concern Value.—(By Town's engineer)		9 000.00

The details are not shown here as they are assumed not to be of any particular interest.

Value and Rating.—Worth of the Service to the Consumer.—Assume profit increment in this case to be 33⅓% and 6% as a fair rate of interest at the date of taking, October 1st, 1907, when boom prices prevailed. Then as based upon reproduction cost as of date of taking, we should have:

Estimated reasonable operating expenses (as per p. 1148)	$2 319.69
Estimated annual depreciation	450.00
6% interest upon reproduction cost ($85 394.36)	5 123.66
6% interest upon profit increment (33⅓% = $28 464.79)	1 707.88
Estimated reasonable annual gross income based upon reproduction cost plus 33⅓% profit increment	$9 601.23

The Company was not quite earning this. Therefore, on this basis, the value would not be reproduction cost ($85 394.36) plus one-third profit increment ($28 464.79) = total $113 859.15, but capitalized value of present income, assuming future growth to offset high-service needs, or

From rate of income at date of taking		$9 364.50
Deduct estimated operating expenses	$2 319.69	
Deduct estimated annual depreciation	450.00	2 769.69
Then estimated net annual income =		$6 594.81
Capitalized at 6% =		109 913.50

Same computation as based upon actual first cost instead of reproduction cost.

Annual operating expenses	$2 319.60
Annual depreciation	345.00
6% interest upon original cost ($65 477.61)	3 928.66
6% interest upon profit increment (33⅓% = $21 825.87)	1 309.55
	$7 902.81

Actual gross income (rate) $9 364.50, therefore, on this basis, the rates are excessive, and, ignoring high-service needs, value would be:

Original cost	$65 477.61
Profit increment (one-third)	21 825.87
Total	$87 303.48

High service was imperatively needed. The Company was considering its need, and had taken preliminary steps toward securing it. The rates warranted it, and would have borne it. The cost of installation was estimated at $17 300, as has been shown. The

allowable profit on this installation would have been $\frac{1}{3} \times \$17\,300 = \$5\,766.66$ on the assumed profit increment basis of one-third. The Company was entitled to a consideration of this profit as a future possibility or probability—not to its full value, as the work had not actually been done, but to a substantial portion of it.

On the other hand, this high service involves extra operating cost, depreciation and fixed charges estimated to amount to $479 per year in excess of anticipated increase in earnings. Therefore, on the reproduction cost basis, no increment in value would result. On the first-cost basis, however, the income would carry it and therefore an additional allowance might be made to the Company of say $5 000 profit (out of $5 766.66) making the total on the original cost basis approximately $87 303 + $5 000 = $92 303.

Summary of Valuation.—

On reproduction-cost basis	$109 913	
On original-cost basis		$92 303
Town's market valuation of real estate (by Company, $7 800)	1 800	1 800
Supplies on hand (agreed value)	1 110	1 110
Total	$112 823	$95 213
Deduct Town's estimate of depreciation to date	3 445	3 445
Net value on reproduction-cost basis.	$109 378	
Net value on original-cost basis		$91 768
Claims of water company		216 000
Award		117 698

As a matter of fact, Town's counsel claimed the following valuation:

Valuation of real estate (by Company, $7 800), by Town		$1 800
Supplies on hand		1 110
All other property		110 000
Gross amount		$112 910
Deduct: Depreciation to date by Town (by Company, $1 800)	$3 445	
Decrease due to fire service	8 000	
Item agreed upon by counsel	235	
		11 690
Net value (by Town)		$101 220

Results to Town of Award of Assumed Value.—

Award, say		$101 220
Cost of fire service		17 300
Investment, exclusive of cost of litigation		$118 520
Annual income:		
Present income	$9 364.50	
New income	986.00	
		$10 350
Operation and interest:		
Operating expenses	$2 769 69	
Operating high service	600.00	
Trustees' salaries	300.00	
Interest, 5% on $118 519	5 926.00	
Sinking fund, 1%	1 185.19	
		$10 781

Any increase of the award, above $101 219.90, will require an increase of present water rates.

Unfortunately, the writer heard but a fragment of the final argument of Counsel for the Water Company and, therefore, is unable to give all the Water Company's figures (except Counsel's valuation of $216 000), but has asked its expert, Walter H. Sawyer, Assoc. M. Am. Soc. C. E., kindly to submit any essential data which may have been omitted, and to correct any inadvertent errors in the statement of the data upon this case made in this paper.

Some Significant Rulings Upon the Admissibility of Evidence by the Appraisers in the Portland (Maine) Water-Works Valuation Case.

In the course of the trial in the Portland water-works valuation suit, some important questions of general interest in water-works valuation were ruled upon by the tribunal, and in view, not only of the magnitude of the case—which covered the valuation of the plants of the Portland Water Company, the Standish Water and Supply Company, the Gorham Water Company, the Foreside Water Company, and the Presumpscot Water Power Company, and amounted to nearly $4 000 000—but also of the excellence of this tribunal (which was made up of Judge Albert R. Savage, of the Supreme Court of Maine, the writer of the Waterville and Brunswick opinions and Chairman of this Board of Appraisers; Judge Frederick A. Powers, recently re-

tired from the Supreme Bench of Maine, who had previously sat as Chairman in the Waterville and in the Livermore Falls cases, and had participated in the Waterville and Brunswick decisions; and United States Marshal Henry W. Mayo, who had himself previously served upon water-works appraisal boards), it may be of interest to quote the more important of these rulings, which may be assumed to be indicative at least of the probable attitude of the Supreme Court of Maine upon the points in question.

These rulings covered the question of admissibility of evidence upon the following issues:

1.—Expert testimony on "going concern" value, "franchise" value, and total value of works. Excluded, except upon purely technical questions involved.

2.—Need and cost of reinforcement of distribution pipe system. Admitted.

3.—Selling prices of water company stock as tending to show value. Excluded.

4.—Depreciation, as based upon or measured by the substitution of another equally efficient or different structure to do the work of the existing structure. Excluded, except as bearing upon relative efficiency of the plant.

The trial of the last three Maine water-works valuation suits, which were all conducted on behalf of the several municipalities by the Hon. Herbert M. Heath, of Augusta, and were unique in American annals of such cases, in that he argued successfully against the admission of any testimony upon the total value of the property of the water company by the experts or engineers, holding that in valuing the property as a whole the expert was usurping the functions of the tribunal; that expert testimony was admissible only in so far as it dealt with facts or matters of opinion beyond the ability of the layman to grasp without expert assistance; and that valuation implied a thorough knowledge of value standards, a knowledge of the law as well as of engineering principles or theories, since valuation was a mixed question of fact, judgment, and the law, and therefore must be made always with a full knowledge of the law upon such cases. The appraisers in all these cases (Kittery, Livermore Falls, and Portland) sustained this contention, as appears in the first ruling, quoted later.

These rulings are the only examples of such a ruling in water-works valuation in America that have thus far come to the attention of the writer. The experts only testified as to engineering facts, principles, and costs, and not as to property values or the elements thereof. In each case, however, the engineers assisted counsel in the preparation of the necessary tables and data upon valuation, which thus went into the arguments, instead of being presented in testimony. The plan here followed would appear to have some advantages, and it seems likely to be tried elsewhere unless exception should be taken to the ruling, be carried up to the full bench, and there be upset on the broad ground of expediency. Abstracts of the rulings follow:

Ruling on Admissibility of Expert Testimony on Going Concern, Franchise Value, and Total Value of Works.

"The Appraisers have taken into consideration the question that has been argued, and I have now to state the result of their conference. Disregarding the precise form of the question which was asked originally, and which we perhaps have forgotten, but taking into consideration the real question which is presented, and which has been argued by counsel, it is, as we understand it, in effect whether the characteristic of going concern values in a case like this can be properly proven by expert testimony. It is not a question whether evidence tending to show a going concern value is material. I do not apprehend that either counsel would take that position; because what is called going concern value is a characteristic of structure, and, if there is any such characteristic, it must be shown by evidence. The only question here is whether the evidence, tendered legitimately, is admissible for that purpose. It may be regarded as presumptuous for a tribunal made up as we are to undertake to illuminate the opinion of the court in the cases in this state which have grown out of proceedings of this character. However, we have—those of us who sat in those cases—gone over the language of those opinions. Mr. Mayo agrees with us that there is nothing in the later case which detracts or modifies anything that was decided in the former case. In the former case the court was asked to say that, in determining the amount to be added to structure value in consideration of the fact that the system is a going concern, 'the Appraisers should consider, among other things, the present efficiency of said system, the length of time necessary to construct the same *de novo,* the time and cost needed, after construction, to develop such new system to the level of the present one in respect to business and income, and the added net incomes and profits, if any, which, by its acquirement as such going concern, would accrue to a purchaser during the time required for such new con-

struction, and for such development of business and income.' And the court said, 'We think this instruction should be given.' The present efficiency of the system, the length of time necessary to construct the same *de novo,* and the time and cost needed, are all matters which affect the characteristic or element, whichever way you may choose to phrase it, of the going concern feature of an existing business.

"Now in the Brunswick case, and I will say right here with regard to the Waterville case, what will also apply to the Brunswick case, that, although the court used the language that the questions before the court, which are comprehensive in scope and minute in detail, in effect relate to the admissibility of evidence, yet they must be decided before the court knows or can know what specific evidence will be offered or relied upon or to what conditions the evidence will be applicable. In such cases it is evident that the answers must be general in character. While it is true that the answers to all these questions do affect the admissibility of evidence, really in effect they laid down rules which made evidence admissible or inadmissible according as it came on one side or the other of the rule. I apprehend that that was substantially what was meant by the court in that expression: 'Specific forms of proof were not under consideration.' How the various elements or characteristics should be proved was not discussed nor decided; and that applies as well to the Brunswick case.

"In the Brunswick case the questions are not printed in full as they were in the Waterville case, but they are incorporated in the opinion, and the first question was, as stated in the opinion, 'We are asked to say that "in applying the rule that the basis of all calculation as to the reasonableness of rates to be charged by a public service corporation is the fair value of the property used by it for the convenience of the public, franchise values are to be wholly disregarded, and the element of going concern value is to be considered only as involved in the structure value; that property value, in this connection, means structure value only."' Then it was that the court thought it was expedient, and perhaps necessary, to lay down some of the principles which affect property of this character. It said that there were three things to be thought of, to be considered, and that none of them could be excluded. They must all be considered. In the end there was a single, final, net result, that none of them could be eliminated from consideration, neither the structure as such, nor the fact that it was in use, nor the fact that there was the right to continue to use it in the place and situation where it was; so that we were unable to give the instruction which we were asked to give for the reasons stated in that opinion. Therefore we feel that we are safe in saying that the two cases are not inconsistent, and that neither is a modification of the other. They discussed propositions from somewhat different standpoints; that is, the questions suggested different ways of approaching them.

"Now we come to the question whether this added element, this enhancement, so far as relates to the going concern values—and I suppose the same ruling applies to franchise values—can be separated out and made the subject of opinion evidence, expert testimony. The question which was asked assumed that a certain number of dollars per annum was a reasonable income. Of course, that assumption is based upon the results of the application of certain rules of law; but there are still other questions of law which, in our judgment, would affect or should affect an answer to a question of that kind, concerning which we have no right to suppose that the witness is qualified to speak. It seems to us that the question of going concern value is material; that it may be shown that in this case, so far as the Company's side is concerned at least all the facts which affect going concern value are already in the case; and that for a witness, no matter what his experience or wisdom to undertake now to derive conclusions from that evidence or state them as conclusions, would be merely stating what appears to him to be the effect of the evidence already introduced in the case. That, as we conceive it, is not the proper scope of opinion evidence. It seems to us that the question is one of argument, one of reasoning conclusions to be derived from the facts in the case. It may be that, in the final consideration of the case, all of the reasons which Mr. Wheeler thinks support his conclusion would be properly presented to us as arguments from the facts proven; that is to say, take the whole case, take the history of the Company, take the number of water takers, take the income—take all of these separate and individual facts which have been declared here one by one, and then how much weight or effect should be given to these facts as affecting the value of the structure because it is a going concern is a matter of argument, of reasoning, and not a matter of opinion evidence; and in that light we exclude the question asked, and shall exclude questions of a similar character.

"It may be summarized in this way: It is not competent for the witness to tell us what his conclusions are from the evidence, but we are the ones to draw our own conclusions after argument."

Ruling upon the Admissibility of Evidence on Need and Cost of Reinforcement of Distribution Pipe System.

"The line which marks the division between testimony admissible with reference to so-called improvements, we think, is pretty fairly well established. The Water District is to take this plant. Comparisons between this and other plants of different construction, differently situated, are not admissible. The District is to take this plant, take it with all its merits and with all its demerits, all its efficiency and inefficiency, if the same is proved. Now if this system is inefficient in any particular, of course, that is a subject of proof which

must be received and weighed by the Appraisers, and in some way the extent of the inefficiency finally to be measured in dollars and cents must be ascertained by proof, and not be left to the Appraisers to guess at.

"Now the questions concerning which the discussion has arisen relate to the efficiency of the fire service within the city of Portland. As we understand it, the Portland Water Campany is now, has been, under certain charter and contractual obligations with reference to the maintenance of a reasonably efficient fire service, that is, to supply water for such a service. That means to supply, not including hydrants or the location of them, but the other means of conveying the water to the various places where it may be needful for the purpose of extinguishing fire. The District takes the plant subject to those obligations. Now it seems to us that if it be true that the system is inefficient, does not come up to the standard which the charter and contract require, that we must in some way know wherein it is inefficient, and how much it is inefficient—not only by having a general statement as to the character of inefficiency, what it fails to do, but what is necessary to make it do what the contract and charter require it to do. That is not figuring on another system, on a new system—it is merely getting at in some way the point wherein this system is said to be lacking. The witness upon the stand has said that in some respects he regards the feature of fire protection as inefficient, and he has stated his standard of efficiency, what he understands and means by the term efficient. Now are we to go along and be satisfied with that unless we know what it lacks, what makes it inefficient? That seems to us to involve further the question of the amount of inefficiency, that is, what it would take to make it efficient, because we cannot measure the inefficiency by guesswork, and in a general sense we think it is true that, when a thing is inefficient, the lessened value by reason of its inefficiency is what it would cost to make it efficient. Suppose, for instance, there were in this case a question about the purity of the water itself, would it not be competent to show how much it would cost us to put in a filtering plant to make it efficient? Would it be proper to show that a filtering plant would do it, and for that reason was necessary to do it, and then how much it would cost? Supposing—that may not be true of this case—that there was a lack of reservoir supply, and that, in order to enable the Company now or the District hereafter to perform its charter and contractual duties, it is necessary to have a larger or a higher reservoir. That is very easy to state, but how are the Appraisers going to know what deduction to make for inefficiency unless they know what it costs to make it? It is this system, not another one, not a new one, but this system brought up to efficiency. We cannot tell without evidence whether the proposed inefficiency may be remedied by putting in a couple of hundred feet of

large pipe at a minimum expense or whether it might cost thousands and thousands of dollars; we do not know anything about it, and we cannot guess at it. We think then that it is competent for the District to show the particulars wherein the present system is inefficient in detail, whether it be the pipe or the outlet from the reservoirs, or wherein it fails to come up to the standard of efficiency. Whether the standard of the witness is too broad is another question. But whether this system comes up to the standard which he as an expert has testified to, we think it is competent for him to say. We must know from evidence something about how much it will cost to put it where it ought to be, if it lacks now in efficiency; and there we think is the line between what is admissible and what is not. With that in mind we admit the question."

* * * * * *

"Excuse me, Mr. Baker, but there is one thing further I intended to say that, while that is admissible, there may be some compensating advantages on the other side which of course the Commission will have to take into account, for instance, for any improvement of the general water system, the Company is entitled to the benefit of so much compensation as is properly derivable from it."

* * * * * *

"Of course the testimony is admitted from the standard which the witness himself sets. If the court thinks that is an extravagant standard, it will have to be taken into account."

Ruling upon the Admissibility of Evidence upon Selling Prices of Water Company Stock as Tending to Show Value.

"The question was argued several days ago, and afterwards we gave the matter not only careful examination in conference among ourselves, but we also carefully examined the authorities which were cited and relied upon. Perhaps it might be sufficient to merely state the conclusion at which we arrived; but, in this case all through we have thought it best to adopt a policy in deciding questions of importance to state the reasons of our rulings. This is only fair to the parties, inasmuch as it is possible that it may not be easy to have our rulings reviewed. Accordingly I will attempt briefly to state the reasons which lead us to the conclusion which we shall announce.

"Of course, evidence of sales of a thing which is under consideration, when the value of that thing is a material issue, is admissible, although ordinarily these are *res inter alios,* admissible because they are the acts of parties having opposing interests, and looking at the value of the thing from different standpoints under circumstances calculated to produce as nearly a correct result in judgment as any human conditions can; and they are supposed to be the result of the

judgment of persons who know, or who assume to know, something about the conditions and quality of the thing sold. If they did not know anything about the essential conditions which entered into the value, the testimony would not be significant or of any importance. Now that is the rule as we understand it or the reasons for it in general. Applying that to this case, or, to lead up to it, first speaking of the matter of authority. The authorities cited by the learned counsel for the Companies seem to divide themselves into three classes. In the first place, as I speak of them, there are a large number of cases where, in tax proceedings or in determining the validity of tax proceedings, the question of franchise valuations was under consideration by the courts, and where the State, seeking to tax the entire value of a corporation, finding that there was a value represented by the entire market value of the stock over and above the value of the plant, the physical value, assessed the difference as being a fair representative of the franchise values. We think that those cases differ very materially from the question which is now presented before us. Of course the State will assess, and should assess, such franchises as the company claims to own, and to be in profitable use of. It will not inquire into the question whether those franchises are used improperly or whether more income is derived from them than should be derived from them under proper tariff rates or prices. If the Company is using them and receiving certain income from them, we apprehend that the State will take the Company's view of the uses which properly may be made when it attempts to assess franchise values. Of course this is a different question from what we have now, where we have to go back of the Company's view of it to get at the real legal rates of income which may be chargeable.

"Then there is another class of cases like the Pennsylvania cases where the court, in discussing values of public service corporations, did state the rule as embracing in evidence the value of the capital stock, the market sales of capital stock as being evidence, not conclusive, but as having some bearing upon the propositions as to the value of the thing itself. We notice that those cases were cases heard and determined, as I remember it, at a date earlier, a time earlier than what may be called the new light on public service values, when the matter had not been so thoroughly studied and differentiated as it has been in more recent years. Those cases make no distinction whatever between public service corporations and other kinds of corporations, nor do they recognize, so far as I remember, the peculiar rules which are now held to attach to the values of public service plants and franchises; and, for that reason, we do not regard them as authoritative—not in this state anyway. But any decision in those days, before the theories were exploited as they have been now, will have to be taken with some considerable question.

"The last class of cases, or the class which embodies perhaps the single case of Smith *v.* Ames, where Judge Brewer did say that in a case involving the unreasonableness of rates charged, that is, whether the rates were confiscatory or not, that the value of the stock was an element to be considered. What Judge Brewer might say about that now in view of more thought, later thought upon the subject, of course no one knows. It was not a question arising on the admissibility of evidence, so that the precise point was sharply presented and argued, but stated by him discursively in his opinion in that case.

"Upon the whole we are of the opinion that we are not concluded by any of the authorities which have been cited; also that we ought not to feel embarrassed in discussing them or considering them in the light of reason.

"When we come to the reason of it, it seems to be something like this: It may be true that if we were to-day valuing an ordinary corporation like a manufacturing corporation, a mercantile corporation, any corporation not engaged in the public service, public utility corporation, it might be that one way of measuring its value over and above the physical value would be to get at the value of its stock sold in the market. The people who bought and sold such stock may be fairly supposed to know what they are buying and selling, and what the rights, what the limitations upon such corporations are. There is no limitation to the amount of money which such corporation is entitled to earn if it can get it; no limitation outside of the charter as to the lines of business in which men engage. All the things which seem to establish the value of such stock may be fairly known, may be fairly presumed to be in the consideration of the men who bought and who sold. But when you come to the matter of a public service corporation, we are met at the very outset with this great difference; that the rights of the corporation to earn money are limited, that is to say, to earn money, to make a charge to its customers. They are limited by rules of law which are not easy, always, of application, they are limited by rules of law which we think are not and cannot be deemed to be within the knowledge and contemplation of investors in its stock. It would be rather a far reach to say that investors even contemplated the physical plant, the most of which is out of sight. Of course this is a matter which would rather go to the weight of the evidence. There are elements in it of law, the application of legal principles which in our judgment remove it from the rule which might perhaps properly be applied to a business corporation; and it is because of this unknown and unknowable element of the legal application of rules to the rate question that we think the testimony is beyond the limit of admissibility. The investor in a property of this kind invests not upon what he sees of the plant, but upon its financial results, always upon the financial results which have already been attained, the dividends now

being earned and declared, the reasonable prospect of business in the future; but that is and must be without regard to the application of legal rules of limitation which we think are beyond, and must be deemed to be beyond, the understanding of the investor, who buys and sells the stock; and for these reasons, after very careful thought and quite an extensive examination, we have all come to the conclusion that the testimony is not admissible."

Ruling on the Admissibility of Evidence on Depreciation Based on the Substitution of Another Equally Efficient but Different Structure to do the Work of the Existing Structure.

"The Appraisers have considered the question submitted, not only at this time, but each of us has given some consideration to the subject since it first arose. The solution of the problem really involves several considerations in the valuing of this plant which is to be taken. One of the elements—and it is only one of the elements—is the present value of the structure, as a structure. That is not all. There are many other considerations which apply to the value of the plant to the Company—its value to them. Among those considerations is the question of efficiency. If the plant is efficient and it can do now all that it is legally and properly bound to do, why, that is one thing. If it cannot do now all that it properly should be required to do, then there arises the element of inefficiency, and if it is inefficient, then it is worth less to the Company to hold and to keep than it would be if it was efficient.

"Now it is in our judgment entirely proper for the taking Company, in introducing evidence so that we can fix the present value, and take into account efficiency, to show wherein it is inefficient, and what it would cost to make it reasonably efficient. That affects the present value of the plant; and we must assume as to the question, as we do assume, that as to efficiency, from that standpoint, that the question of admissibility must be considered. They say it is inefficient and they have a right to prove it if they can.

"Now, Mr. Hazen has pointed out as an engineer the features of inefficiency. He has been permitted to show wherein and how the present system may be made efficient to-day or on the first day of January, the time when the taking took place; and the cost of making it efficient on that day—not taking properly the cost of making it efficient for some future demands and future growth, but efficient as of the first day of January. Now it may be that, in making it efficient on that day to meet present demands, that the results—the beneficial and pecuniary results may be very largely in excess of the cost of making the enlargement of improvement, the enlargement and improvement limited to making it efficient on the first day of January—not in regard to future demands.

"Now, Mr. Hazen says, as we understand it, that in going over this problem and studying it—at least so far as one of the elements that enter into this taking is concerned—he has considered the business proposition, the economical proposition from the standpoint of whoever may be faced with the practical problem. He says that, while a certain arrangement or certain things may to-day be such that practical business men would not do it that way because it involves so much waste and loss in view of future demands that may be made, so he has taken into account, as he says, the business problem. He assumes that this work should be done practically now—that is covering the period of two years to do it, but making five years to cover contingencies to make it practical; and upon that assumption, or business assumption, he has introduced what seems to us to be a new system so far as the supply part of the property is concerned, and that after five years the whole system would become salvage, what is left of it, and that there will be a new pipe or should be for the business standpoint of very much larger capacity than the present pipe, in a new location and under new conditions. And taking that as one of his elements he reasons out upon his statement what the results would be from an engineering and a pecuniary standpoint—economical standpoint.

"Now, of course, Mr. Hazen is entitled to give his conclusions as to efficiency and his reasons for them. Whether the reasons are sound or fallacious does not affect their admissibility. If they are fallacious they can probably be shown to be so, and if they are not they would stand. But whether his theory involves elements not proper to be considered in arriving at the result is a question to be considered respecting its admissibility, and we think that the introducing of what is practically a new system of supply goes beyond the limit, because, as he says, at the end of five years it would be a new and larger supply system, which opens a door into the future to such an extent that the testimony in our judgment is not only improper but goes beyond the line of legal admissibility.

"The record, as it comes from the stenographer's hands, does not show the precise question which was involved, because the witness was asked to turn to his table and that does not appear in the record, but we have the table as passed to us in the examination and assume he was to explain his views upon this table as to the mathematical result at which he had arrived from his system of computation; and that, as presented, we exclude."

Effort to Eliminate, in Water-Works Valuation, the Value of Franchise and Other Rights Acquired by Legislative Grant.

With the growing recognition of the rights of the public, and the obligations, as well as the rights of public service corporations, has come the endeavor on the part of the public to retain for itself, in

granting authority to operate quasi-municipal corporations, any values that may develop directly from these grants.

Thus Massachusetts has sought to eliminate the element of franchise value in water-works valuation by express provision in the Charter or Incorporation Act of the Companies, covering this point; hence in the Haverhill, Newburyport, and Gloucester cases (as in the Kansas City case), franchise considerations were eliminated and the works were appraised upon the basis of their value to the purchasers rather than to the sellers, as is provided for in the Maine rulings.

A somewhat similar situation is found in England, as outlined in the report of the London County Council upon the "Water Supply of London, and a Bibliography of the Principal Reports and Papers on the Subject, Together with a Special Report by the Comptroller of the Council on the Financial Aspects of the Purchase."*

English Precedent for Restricting the Valuation of the Franchise and Other Rights Acquired by Grant.

London's water supply dates back to the 13th century, when the Corporation of the City of London obtained leave to construct conduits for the purpose of bringing water from Tyburn to the City "for the profit of the City, and good of the whole realm thither repairing; to wit, for the poor to drink, and the rich to dress their meat." The system was more or less extended until, in 1580, the first rights were granted for supplying water to the public through private undertaking. From that time rights were granted at long intervals of time to additional water companies, until there were eight or perhaps more water companies supplying the City of London in the early part of the last century.

The service rendered by these companies proved inadequate, as was shown by the comment of Sir George Grey, in introducing his Parliamentary Bill upon this subject in 1851, that "competition could not be safely trusted to secure an adequate supply of good and wholesome water which was essential to the health and comfort of the inhabitants of the metropolis," and led to growing discontent on the part of the public. This resulted in the formation of a series of Committees of Parliament and County Council for the consideration of the public

*Printed for the London County Council by P. S. King & Son, in June, 1905.

control of the water supply, extending from 1821 to 1902, in which was passed the "Metropolis Water Act," "for the purpose of acquiring by purchase and of managing and carrying on the undertakings," of the water companies, and "generally for the purpose of supplying water" within an area differing in some respects from the limits of actual and possible supply of the companies.

As far back as 1891, the London County Council reported, among other things:

"2. * * * That for certain purposes at least it would be desirable to acquire the undertakings of the existing companies if the same could be obtained on fair and reasonable terms.

"3. That the Council is of opinion that the price to be given for the undertaking of any company should depend, not merely on past dividends or on Stock Exchange values, but upon the true value of the undertaking, having regard to its legal position and liabilities; to the condition of the property; and to its ability to supply future wants.

"4. That, in order to arrive at a satisfactory conclusion on this last question, it is essential, before any terms of purchase can be considered, that the liabilities to capital expenditure for new or supplementary sources of supply in the immediate future should be authoritatively ascertained."

The works were finally taken on a basis of their fair and reasonable value.

In the course of this long-protracted struggle between the public and the water companies, the restrictions under which Parliamentary grants were made to the water companies, permitting them to extend their facilities and service, became more and more drastic from the companies' point of view, and protected the rights of the city more and more carefully from the public point of view. Thus in the Act of 1847, it was provided that:

"The profits of the undertaking to be divided among the undertakers in any year shall not exceed the prescribed rate, or where no rate is prescribed they shall not exceed the rate of ten pounds in the hundred by the year on the paid-up capital in the undertaking, which in such case shall be deemed the prescribed rate, unless a larger dividend be at any time necessary to make up the deficiency of any previous dividend which shall have fallen short of the said yearly rate."

In 1878 Parliament introduced:

"Into the Grand Junction Company's Act of that year, authorizing the raising of fresh capital, what are known as the Auction Clauses,

which were designed to secure the economical raising of capital by compelling the companies to offer the stock for sale by public auction or tender."

In 1886 was passed the Sinking Fund Clause:

"The object of which was to secure for the public, by means of transfer to a Trust Fund, some share in the increasing profits of the undertakings.

"The effect of the section was that, after three years from the issue of any debenture stock under the Acts, the companies had to pay each year to the Chamberlain of the City of London the difference between the profits earned by the stock as part of the entire capital of the companies and the interest actually paid to the holders of the stock plus 1 per cent., which was allowed to the companies for 'management.'

"This allowance of 1 per cent. was proposed by the City Corporation as a sort of margin to the companies for managing the business. As a matter of fact, inasmuch as all the expenses of management were necessarily charged against profits before ascertaining the proportion of profits attributable to the new debenture stock, the 1 per cent. was really a bonus or present to the companies concerned."

In some of the later Acts, as in the Staines Reservoir Act of 1896, and the Kent Co. Act of 1902, a Sterlization Section was inserted as follows:

"Nothing in this Act shall authorize the company to bring into account or to make any claim in respect of any advantages conferred on them by or resulting from the passing of this Act. Provided that in the event of such purchase the company may bring into account the actual amount of any capital expenditure made by them in exercise of the powers of this Act.*

"The sinking fund and sterilization clauses were designed to secure in part the same object, viz., the non-enhancement of the value of a company's undertaking in the event of purchase by reason of the further advantages given to the company by the special Act, but they differed in important respects as regards the manner and extent of their operation. The sinking fund attached only to capital powers authorized and directly affected the profits of the company by appropriating annually in perpetuity a portion of such profits for the benefit of the public. The sterilization clause, on the other hand, was directed solely to the contingency of purchase, and its operation was limited to a particular period of years; it was in general terms and could be applied both in the direction of reducing the company's future profits for the purpose of purchase, and as affecting the number of years'

* Extract from Section 20, Kent Co. Act of 1902.

purchase, because its effect was to deprive the company of any advantages conferred by, or resulting from, the passing of the special Act.

"The importance of this legislation can hardly be over-rated, but the companies were purchased before its full effect had been felt."

All as stated by the Comptroller of the London County Council in his report.

The valuation of the London companies appears to have been based upon a consideration of:

1.—The dividend being earned by them upon the capital stock of the company which is restricted rigidly to actual cash investment; allowance being made for a 10% dividend from the time of incorporation of the company, provided the income of the company justified this return.

2.—The probable continuity of and growth in future income.

3.—The character of the plant and of the service furnished and the probable needs of the future.

4.—The effect of the sinking fund, sterilization, and other similar clauses under which the companies were operating.

5.—A 1½% allowance for cost of reinvestment and other costs.

As stated by the Comptroller, the company's works were valued upon a basis of their value to the vendor, rather than to the purchaser; upon the "perpetual existence of the companies, and apparently upon the adoption of an average of the past ten years as a starting point for their valuation."

The Comptroller further observes that he doubts "whether sufficient weight had been attached in the recent arbitrations to the large expenditure on capital account which will be required at no distant date."

It is also interesting to note that, as in the recent Maine cases, "the Court ruled that the question of the value of the undertaking was a matter for the Court, and his evidence (the witness') was therefore limited to facts concerning the condition of the Company's works and plant."

It is impossible, within the limits of this paper, to discuss in detail the awards in the London cases, but those particularly interested in this subject will find the report of the London County Council, referred to, well worth perusal.

TABLE 4.—Some Recent Awards in New England in Water-Works Valuation Cases.

Date of taking.	Name.	Year of original construction.	Population.	Estimated actual cost.	Reproduction cost. Approximate.	Depreciation to date.	Approximate net annual income, exclusive of depreciation and interest charges.	Award.	Award per capita.	Ratio of award to reproduction cost.	Ratio of net annual income to award without allowance for depreciation. %
June 12, 1887	Braintree, Mass.	.....	4 300 (1)	.....	.....	.....	$10 750 (2)	$159 610.44	$37.12	.....	6.7
Oct., 1895	Gloucester, Mass.	.....	28 211	.....	.....	.....	.....	600 500.00	21.29	.....	....
1898	Fulton and Oswego Falls, N. Y.	.....	.....	.....	.....	.....	11 340	172 500.00	.....	.....	6.6
1901	Newmarket, N. H.	.....	1 800	.....	.....	.....	5 100	83 000.00	46.11	.....	6.2
1902	Dover and Foxcroft, Me.	.....	3 600 (1)	.....	.....	.....	{ 7 057 (3) { 7 500 (4)	{ 135 659.31	37.70	.....	{ 5.2 { 5.5
1903	Gardner, Mass.	.....	11 700 (1)	.....	$165 000.00 (5)	....	{ 13 700 (6) { 20 000	{ 274 000.00	23.42	1.66	{ 5.0 { 7.3
July 1, 1903	Gardiner, Me.	.....	7 380 (1)	.....	132 000.00 (7)	$35 854 (7)	15 556	245 000.00	33.20	1.85	6.4
July, 1903	Waterville, Me.	1887	17 600	$250 000	368 546.00 (8) 288 776.00 (7)	32 400 61 800	32 700 (9)	.....	.....	1.37 1.74	6.5 6.9
Jan. 1, 1904	Brunswick, Me.	.....	6 930	.....	130 455(24) (7)	18 529 (7)	15 000 (25)	196 162.09	28.31	1.24 (24)	7.7
1904	Augusta, Me.	.....	12 100 (1)	300 000	.....	.....	31 000	427 135.20	35.29	1.5	7.3
Jan. 1, 1905	Athol, Mass.	1877 (?)	7 196	.....	206 386.00	39 253 (7)	17 000	316 500.00	43.99	1.53	5.4
1905	Amesbury, Mass.	.....	8 840	.....	205 573.00	46 521	13 000	265 000.00	29.98	1.29	4.9
Dec., 1906	No. Conway, N. H.	1884	1 700 (12)	.....	35 300.00 (13)	8 100 (14)	3 700	62 000.00	36.46	1.76	6.0
1906	Ithaca, N. Y.	1872	18 000 (15)	.....	{ 551 575.00 (16) { 322 793.00 (17)	.....	40 750	658 000.00	36.56	{ 1.19 { 2.04	{ 6.2
Jan. 1, 1907	Derry, N. H.	1891	4 500	.....	80 878.06	12 750	7 399	125 500.00	27.89	1.55	5.9
July, 1907	Kittery, Me.	1902	3 000 (18)	81 000	102 510.25	2 200	10 786 (19)	163 869.00	54.70 (18)	1.60	6.6
Oct., 1907	Livermore Falls, Me.	1899	3 142	66 000	85 394.36	{ 3 455 (20) { 1 800	{ 9 364	117 698.00	37.46	1.38	8.0
Jan. 1, 1908	Portland, Me.:										
	Portland Water Co.	{ 1868 { *et seq.*	{ 45 600	1 425 500	1 525 476.00 (21) 1 685 696.00 (22)	656 694 (23) 84 609 (8)	..... 147 055	.....	.....	.....	.....
	Gorham Water Co.	1895	2 500	.....	67 000.00 (21) 71 960.00 (22)	5 856 (23) 4 504 (8)	..... 2 291	2 539 235.00	51.20	1.53	5.9
	Foreside Water Co.	1906	1 500	58 300	70 400.00 (21) 71 920.00 (22)	649 (22) 212 (8)	 0	.....	.....	.....	.....
	Standish Water and Constr. Co.	1888and earlier.	28 600	.....	931 500.00 (21) 1 024 120.00 (22)	233 394 (23) 35 014 (8)	64 200	1 313 798.00	45.94	1.41	4.9
	Average of column								$36.40*	1.54	†6.2
	Average based upon totals								37.52*	1.50	†5.9

* Excluding Fulton and Oswego Falls. † Excluding Gloucester.

Some Recent Awards in New England, in Water-Works Valuation Cases.

Table 4, though more or less fragmentary, may be of casual interest to the reader, particularly if he is familiar with the works. Without a fair knowledge of local conditions, it is not safe to generalize from figures of this kind. Particular attention is called to the fact that no deduction has been made from the net annual income to cover depreciation or interest upon fixed charges, and similarly in determining the ratio of net income to award, this ratio is determined from the approximate net annual income exclusive of depreciation and interest charges as shown.

Notes to Table 4.—

(1) Estimated from Census.
(2) Charles F. Parks' estimate of future income, including estimated hydrant rental.
(3) Normal for 1902, estimate of expert for company.
(4) Anticipated for 1903, estimate of expert for company.
(5) Finding of Commission after deducting depreciation on value of physical plant, exclusive water rights, street franchises, and going concern value.
(6) Finding of Commission on fair net or divisible income.
(7) F. C. Coffin, expert for town.
(8) William Wheeler, expert for company.
(9) Estimated by Water District Engineers.
(12) Exclusive of large summer population.
(13) Agreed reproduction cost exclusive of engineering and contingencies $31 380. Amount given includes 12½ per cent.
(14) Town experts' estimate.
(15) Exclusive of 3 000 at University.
(16) Average of company's experts, after deducting depreciation.
(17) Average of city's experts after deducting depreciation.
(18) Bulk of water used at Navy Yard for manufacturing.
(19) $10 346 less depreciation.
(20) (?)
(21) Exclusive of service pipes, hydrant, and drinking fountains.
*(22) Inclusive of service pipes, hydrant, and drinking fountains.

* Ownership at issue, and probably vested in the water consumers.

(23) Exclusive services, etc., Water District experts' estimates.

(24) Agreed reproduction cost by engineers, $158 389.40.

(25) Average net revenue, 5½ years, $14 434.40.

Water-Works Valuation.

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Conclusion.

It seems unnecessary to attempt a recapitulation of the matters discussed herein, as the length of this article has far exceeded intended limits, and as only comparatively few directly interested in water-works valuation and fair-rates cases will be likely to scan these lines. The paper was prompted by the desire to submit, for discussion, by members of this Society, the method outlined herein of applying, in water-works valuation, the vital principle or gauge of fair rates—"the-worth-of-the-service-to-the-consumer," so admirably laid down by Judge Savage in the Brunswick decision—a decision destined to be much quoted, and to exercise a large influence in water-works valuation and rating cases in the future.

It is not suggested that the method discussed and illustrated herein is the only correct method, or the final criterion of value or that it is the interpretation which the Supreme Court of Maine would put upon its own ruling. It furnishes, however, another standard for water-works valuation and for the determination of fair rates—perhaps

a more tangible and rational one than some that have been used in the past.

As applied to works which have been seriously over-capitalized to cover up excessive earnings, it will bear heavily; to capital actually and prudently invested in the property, it will afford ample protection.

To old and outgrown works which, not content with dividing the real earnings of the company—and failing to recognize and provide for the inevitable depreciation resulting from lapse of time, long service, and other conditions—have actually divided up the plant in dividends, it will result in present loss to an over-greedy corporation. To the prudent and well-managed company and to new capital seeking opportunity for investment in public service works, it will give added safety and be attractive.

In conclusion, the writer desires to make grateful acknowledgment of his obligation to Messrs. Allen Hazen, William Wheeler, Harrison P. Eddy and Charles W. Sherman, Members, Am. Soc. C. E., for friendly suggestions upon this paper, and to the Hon. Herbert M. Heath, of Augusta, Maine, Corporation Counsel, for permission to make use of his arguments in the Maine case cited.

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PAPERS AND DISCUSSIONS

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THE IMPROVEMENT OF THE OHIO RIVER.

By William L. Sibert, M. Am. Soc. C. E.*
To be Presented December 16th, 1908.

The recent completion of Locks and Dams Nos. 2, 3, 4, and 5, Ohio River, creating, in connection with Locks Nos. 1 and 6, a navigable depth of 9 ft. from Pittsburg to Beaver, Pa., a distance of about 30 miles, and constituting the first completed section of the 9-ft. project, caused the writer to think that a review of the various engineering projects proposed for the improvement of this river, might be of interest to the profession.

The writer had charge of the construction of Locks Nos. 2, 3, 4, 5, and 6, from 1903 to the spring of 1907. Captain E. N. Johnston, Corps of Engineers, U. S. A., one of his assistants at that time, aided materially in preparing this paper.

Physical Characteristics.

The Ohio River is formed, at Pittsburg, by the junction of the Allegheny and the Monongahela Rivers. Its total length, all of which is navigable, is 967 miles. The navigable length of its tributaries is about 3 000 miles. The drainage area above Pittsburg, including the

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Major, Corps of Engineers, U. S. A.

valleys of the Allegheny and the Monongahela, is about 19 000 sq. miles, while the entire water-shed comprises an area of about 214 000 sq. miles, which is larger than the area drained by the Mississippi River above the mouth of the Missouri, and larger than the drainage area of any other river in the United States, except the Missouri and the Lower Mississippi. The total fall at low water from Pittsburg to Cairo is 426 ft. In the first 26 miles of the river, from Pittsburg to Beaver, the average slope is about 15 in. per mile; from Beaver to Wheeling, a distance of 64 miles, 9 in. per mile; from Wheeling to Louisville, a distance of 509 miles, 5½ in. per mile; while from Louisville to Cairo, the slope is only 4 in. per mile.

In its upper portion, the river is subject to frequent and rapid fluctuations of water surface.

In connection with the generally accepted theory that the destruction of the forests has increased the number and extent of floods, and, at the same time, decreased the low-water flow of the streams, the following statistics are cited:

Records of the river stages in Pittsburg for the 24 years preceding 1881, show the following:

River Stage.	Average Number of Days per Year
½ to 1 ft.	½ day.
1 to 2 ft.	34 days.
2 to 3 ft.	37 days.
3 to 4 ft.	47 days.
4 to 5 ft.	43½ days.
5 to 6 ft.	40 days.

Average number of days when river was below 6 ft., 214.

Average number of days when river was above 6 ft., 151.

For the 24 years succeeding 1881:

Average number of days when river was below 6 ft., 201.

Average number of days when river was above 6 ft., 164.

In Colonel Ellet's book, "The Ohio and Mississippi Rivers," the following record of lowest water depths on the bar at Wheeling is found:

Sept. 27, 1838	0 ft. 10½ in.	Sept. 19, 1845	2 ft. 2 in.
Sept. 6, 1841	1 " 0 "	Oct. 13, 1846	1 " 9 "
Aug. 17, 1843	1 " 8 "	Sept. 8, 1847	2 " 3 "
Sept. 23, 1844	1 " 1½ "	Sept. 18, 1848	1 " 11 "

The number of times that the Ohio River was above the danger stage at Pittsburg, Cincinnati, and Louisville, during the sixteen years preceding 1891 and the seventeen years following, is shown in Table 1:

TABLE 1.

Place.	Height of danger line above zero of gauge, in feet.	First period, 16 years, 1875-1890, inclusive. Times.	Second period, 17 years, 1891-1907, inclusive. Times.
Pittsburg, Pa.	22	18	26
Cincinnati, Ohio.	50	11	11
Louisville, Ky.	28	11	9

It is to be expected that the records at Pittsburg would show a greater number of river stages above 22 ft. in the years succeeding 1890 than in the preceding years, because of the material contraction of the river channels at that place due to encroachments of railroads and manufacturing plants. However, the conditions at Louisville and Cincinnati have changed so little that fair conclusions can be drawn from the records at those places.

While the highest gauge readings of which there is record in Pittsburg occurred March 15th, 1907, on which date a stage of 35.6 ft. was reached, it is thought that during the flood of 1832, which reached a height of 34.94 ft. at Pittsburg, the discharge of the river was probably equal to that of March 15th, 1907. A comparison of the old and present maps shows a marked contraction of the river channels near Pittsburg on account of slag dumped over the banks into the streams.

It seems to follow from these data that the extremes as to flood height are not materially influenced by forests, but that the frequency of medium size and possibly destructive floods may be increased by deforestation. These data further show that the deforestation of the Ohio water-shed has had practically no effect upon the extent of the low-water navigation seasons.

A theory has been advanced that the low-water stages of rivers are less dependent upon springs than upon summer rains, first on one tributary and then on another. This has been especially noted on the Monongahela River, which stream becomes exceedingly low during the summer and fall months, unless there be local rains, notwithstanding

a copious prior winter and spring rainy season. Summer rains, if the lands be denuded of forests, reach the river; whereas, if a forest exists, the rain is absorbed by the thirsty plant life, or evaporated.

The maximum recorded discharge of the Ohio River just below Pittsburg, is 439 000 cu. ft. per sec. The discharge at Davis Island Dam, just below Pittsburg, has been observed in recent years to be as small as 1 600 cu. ft. per sec. Captain Saunders gauged the Ohio River at Pittsburg in 1838, during the drought, and found the discharge to be 1 400 cu. ft. per sec. The quantity of water emptied into the Ohio by its tributaries during the dry season is very small, the low-water discharge of the Monongahela River being only 166 cu. ft. per sec. Although the low-water discharge of practically all the tributaries is too small, and the slopes of many of them are too steep to admit of an efficient improvement by regularization, the discharge is large enough to permit an extensive commerce to be carried on through a system of locks and dams, that on the Monongahela River being about 12 000 000 tons per year. It is not thought that such an extensive use of this or other streams of similar slope would be possible unless the same were canalized, this assuming the existence of sufficient water to make the needed open river channel depths. The present system of dams with movable tops will create a navigable depth of about 11 ft.

Even during the dryest periods, an abundance of water exists in the Ohio for the maintenance of a 9-ft. depth in the pools of a slack-water system, as has been demonstrated at Davis Island Dam built more than twenty years ago.

Usually, the low-water period includes July, August, September, October, and November. During the low-water season, the depth of water on the bars near Pittsburg is from 12 to 18 in.; sometimes for nearly three months the bar depths are less than 2 ft. The lack of navigable depth in the lower river is caused largely by the excessive width of the channel.

Table 2 shows the number of days, within the period named, that a 9-ft. stage or more, existed at important points along the river.

The present project for improving the Ohio contemplates a navigable depth of 9 ft., which project, without changing any structures, could easily provide for 11-ft. navigation by dredging in the upper end of the pools. It will be observed from Table 2 that the number of days per year during which there is a natural stage of 9 ft. or more, in-

creases as the river is descended. From Pittsburg to Beaver, the mean number of days that a 9-ft. navigation is afforded is eighty-one. The Pittsburg District, including a radius of 50 miles around the city proper, is the most prolific freight-producing center in the world. The intermittent navigable stages in the river, aggregating only 81 days per year, has debarred this district from an extended use of the Ohio, except for the cheaper grades of freight such as coal, and for that only where a long haul is necessary.

TABLE 2.

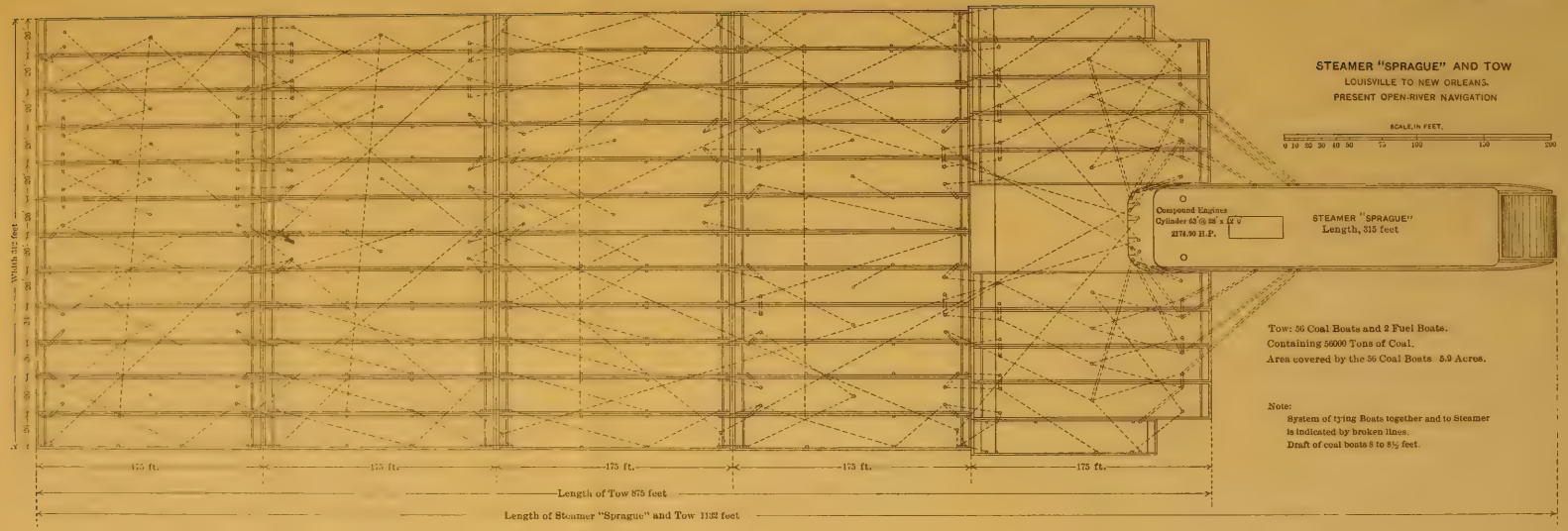
Place.	Miles.	Mean number of days at or above 9 ft. during the 10 years, 1895–1905.	
Pittsburg, Pa.	0.0	81.0	
Dam No. 6.	28.5	117.0	
Wheeling, W. Va.	90.0	119.2	
Parkersburg, W. Va.	183.5	140.5	
Point Pleasant, W. Va.	263.4	150.3	
Portsmouth, Ohio*	353.0	216.4	
Cincinnati, Ohio.	466.5	248.3	
Madison, Ind.*	553.5	233.0	
Louisville, Ky.	599.0	97.5	Head of Falls.
Evansville, Ind.	783.0	198.5	
Paducah, Ky.*	919.5	203.6	
Cairo, Ill.	967.0	302.4	

* Five years' records only.

During the periods that intervene between freshets, a great quantity of freight is kept in the harbor at Pittsburg awaiting a rise. The steamboats are tied up, much capital is idle, and great expense is incurred in watching the boats and barges, pumping them out, etc.

"In June, 1895, there were collected in the Pittsburg harbor, 1 200 000 tons of coal, loaded upon about 2 500 vessels awaiting water to move them down the Ohio River, the largest tonnage ever assembled in any harbor of the world at any one time. The rise did not come until November 27th. The cost of freight and vessels engaged in this service was estimated at $6 310 000. It cost $2 000 per day to keep the tonnage afloat, and $1 000 per day interest on the investment. Total, $3 000 per day. This tonnage was kept waiting in the Pittsburg harbor for water in the Ohio River an average time of five months, or 150 days, at a loss of $450 000, which is 5% of $9 000 000. This shows what one item of commerce lost in five months, because Western Pennsylvania did not have the economy of transportation that results from continuous water movement."*

* Address by Mr. John E. Shaw, before the Merchants' and Manufacturers' Association at Pittsburg.


STEAMER "SPRAGUE" AND TOW
LOUISVILLE TO NEW ORLEANS.
PRESENT OPEN-RIVER NAVIGATION
SCALE, IN FEET.
0 10 25 30 40 50 75 100 150 200
Width 312 feet
Compound Engines
Cylinder 63 & 98 x 10'0
2174.90 H.P.
STEAMER "SPRAGUE"
Length, 315 feet
175 ft.
175 ft.
175 ft.
175 ft.
175 ft.
Length of Tow 875 feet
Length of Steamer "Sprague" and Tow 1132 feet
Tow: 56 Coal Boats and 2 Fuel Boats.
Containing 56000 Tons of Coal.
Area covered by the 56 Coal Boats 5.9 Acres.
Note:
System of tying Boats together and to Steamer is indicated by broken lines.
Draft of coal boats 8 to 8½ feet.

Boats and Commerce.

Boats suitable for navigating the Ohio River do not anchor, but are tied to the shore, and a sloping shore is needed so that they may be moved toward the bank as the water rises, thus keeping out of the stronger currents where drift would lodge against them and break them loose.

The short-lived freshets of the Upper Ohio have developed an unique system of towing on the Ohio and Mississippi Rivers. At times of freshets, freight is carried in great fleets made up of boats and barges. The boats are about 26 ft. wide by about 175 ft. long, and each carries approximately 1 000 tons. The barges are 26 ft. wide by about 130 ft. long, and each carries about 500 tons. As they leave the upper river, the fleets vary in size, containing from 10 to 20 boats or barges. These fleets are increased in size as they proceed down the river, and sometimes comprise from 50 to 60 boats of 1 000 tons each, the fleet covering 5 or 6 acres of water.

Plate CXXVII shows the Steamer *Sprague* and a tow of coal barges.

When loaded, coal barges draw 6 ft., and coal boats from 8 to 9 ft., the former requiring a navigable depth of 8 ft., and the latter from 10 to 12 ft. The excess depth of from 2 to 3 ft. is needed, because the large fleets generally require a channel width of at least 300 ft. of the full depth of the deepest laden craft in the fleet, and to obtain this it is generally necessary to have a river stage at least 2 ft. higher than that necessary for a single boat. A slack-water depth of 9 ft. is at least equal to an 11-ft. open-river depth. With slack water, the pools are more than 9 ft. deep at all places except immediately below the locks. There being no currents to interfere, a little dredging allows fleets of full depth and width to pass into the deeper water and wider channels further down in the pools. The absence of currents greatly assists navigation in keeping in narrow channels.

The steamboats are of the stern-wheeled type. Experience has shown that these boats are exceedingly efficient for the work required. They are equipped with long balanced rudders against which the water from the wheel impinges with high velocity when the boats back, thus driving the stern in the desired direction whether the boat has headway or not. With this device, boats of 2 000 h. p. can successfully maneuver, down stream in swift water, fleets of from 40 000 to 60 000 tons burden.

In a report submitted to the Chief of Engineers, in 1870, the late W. Milnor Roberts, Past-President, Am. Soc. C. E., stated that:

"A system which will provide uninterrupted navigation will undoubtedly revolutionize the system of coal carriage. Less powerful steamers can then be used steadily all the year round, making regular trips, each boat taking fewer barges, but doing the same, or more business, with less capital invested at less risk, and without the expense due to idle boats and the expense of watching the great fleets while awaiting a freshet."

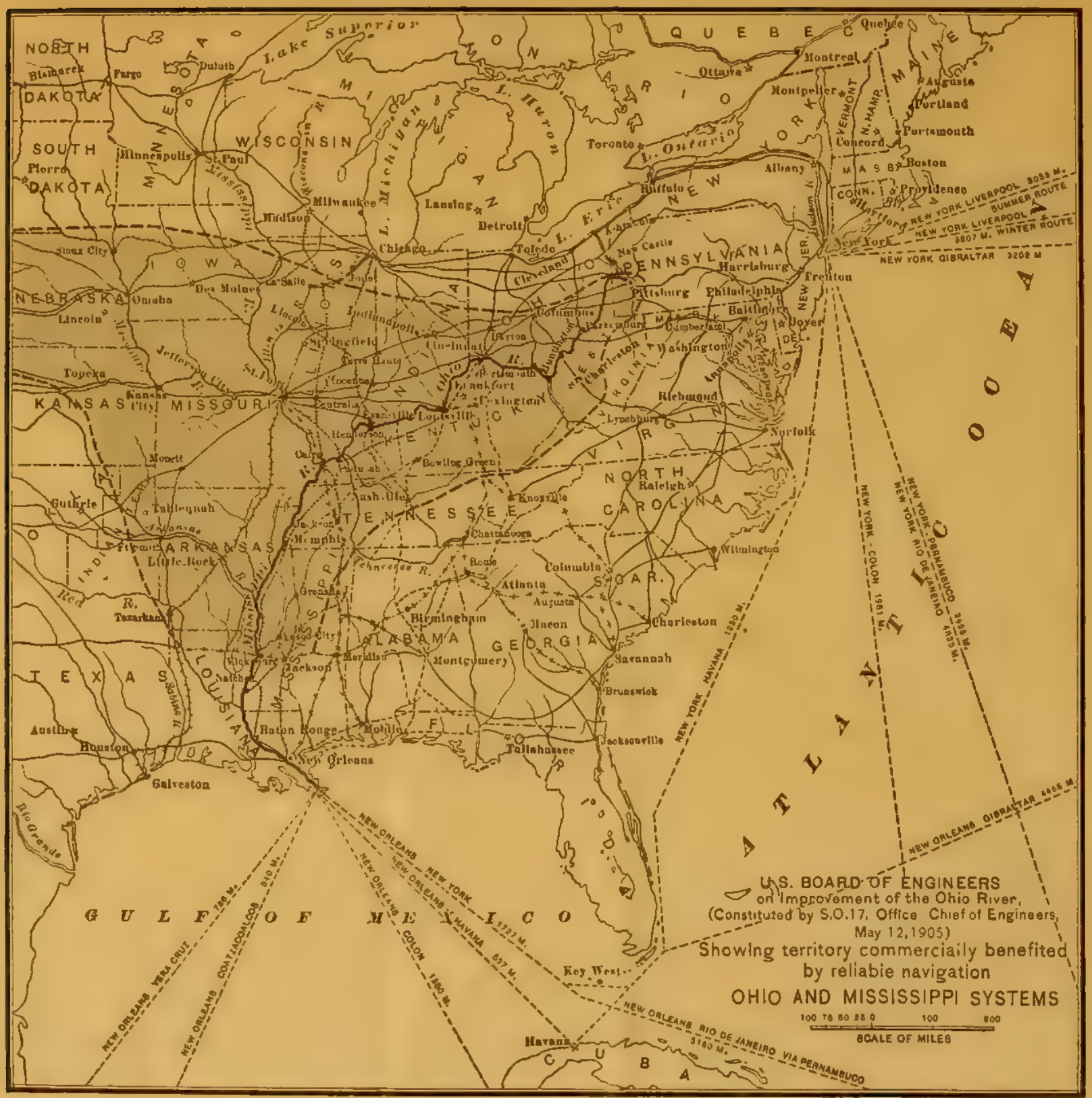
A system of locks and dams was referred to by Mr. Roberts in the paragraph quoted.

The records of freight cost under the present towing system, together with the estimated cost with a slack-watered Ohio, indicate that the export freight of the Ohio Valley, offered in barge-load lots, should be delivered at New Orleans at about $1 per ton.* Boats could then advantageously transport freight up and down stream. In an unimproved river, or in one improved by regularization, currents are often a serious handicap to up-stream navigation.

By adding to the foregoing cost a reasonable water rate from New Orleans to San Francisco, including a toll charge of 75 cents per ton through the Panama Canal, it appears that, with a slack-watered Ohio and a completed Panama Canal, freight should be transported from the Ohio-Mississippi water-shed to the Pacific Coast at from $5 to $8.50 per ton. With the present class rates by rail from Chicago or New York to San Francisco at from $19 to $32 per ton, and with commodity rates at about 4.3 mills per ton-mile, one would expect a large commerce, *via* the all-water route, between the people of the Ohio-Mississippi water-shed and those living between the Rockies and the Pacific Ocean. The Panama Canal will, of course, offer similar opportunities to the Atlantic Coast, both as to trade with the Pacific Coast and with China and Japan. In this connection the following is inserted:

"Recently 2 500 tons of steel rails were shipped from Pittsburg for the construction of the Prince Rupert and of the Grand Trunk Pacific System, by what, on its face, appears to be an exceedingly roundabout route. The rails were first sent to New York, where they were loaded on vessels going to the Orient by the Suez Canal Route. On reaching Kobe they will be transshipped and sent across the Pacific to their

* Executive Doc. 492, 60th Cong., 1st Sess., pp. 33 to 35.


U.S. BOARD OF ENGINEERS
on Improvement of the Ohio River,
(Constituted by S.O.17, Office Chief of Engineers, May 12,1905)
Showing territory commercially benefited by reliable navigation
OHIO AND MISSISSIPPI SYSTEMS
SCALE OF MILES
NEW YORK LIVERPOOL SUMMER ROUTE 3055 M.
NEW YORK LIVERPOOL WINTER ROUTE 3207 M.
NEW YORK GIBRALTAR 3202 M
NEW ORLEANS GIBRALTAR 4466 M.
NEW YORK HAVANA 1330 M.
NEW YORK - COLON 1961 M.
NEW YORK - PERNAMBUCO 3566 M.
NEW YORK RIO DE JANEIRO 4925 M.
NEW ORLEANS - NEW YORK 1727 M.
NEW ORLEANS HAVANA 837 M.
NEW ORLEANS COLON 1880 M.
NEW ORLEANS COATZACOALCOS 810 M.
NEW ORLEANS VERA CRUZ 788 M.
NEW ORLEANS RIO DE JANEIRO VIA PERNAMBUCO 5160 M.
ATLANTIC OCEAN
GULF OF MEXICO
CUBA
Havana
Key West
Galveston
New Orleans
Mobile
Jacksonville
Tallahassee
Savannah
Charleston
Wilmington
Norfolk
Richmond
Washington
Baltimore
Philadelphia
Harrisburg
Pittsburg
Cleveland
Toledo
Detroit
Buffalo
Albany
Boston
Portland
Montreal
Quebec
Toronto
Chicago
Milwaukee
Minneapolis
St. Paul
Duluth
Fargo
Bismarck
Pierre
Sioux City
Omaha
Lincoln
Topeka
Kansas City
St. Louis
Memphis
Vicksburg
Natchez
Shreveport
Little Rock
Fulton
Texarkana
Galveston
Houston
Austin
Monett
Guthrie
Tahlequah
Paducah
Cairo
Evansville
Indianapolis
Columbus
Cincinnati
Louisville
Frankfort
Lexington
Bowling Green
Nashville
Knoxville
Chattanooga
Atlanta
Augusta
Macon
Columbia
Raleigh
Lynchburg
Parkersburg
Wheeling
Pittsburg
Erie
Lansing
Madison
La Salle
Des Moines
Dubuque
Davenport
Rock Island
Peoria
Springfield
Terre Haute
Vincennes
Evansville
Grenada
Meridian
Montgomery
Birmingham
Jackson
Baton Rouge
Portsmouth
Concord
Hartford
Providence
New York
Trenton
Dover
Wilmington
Cumberland
Harrisburg
Ottawa
Montpelier
Augusta
Portland
Portsmouth
Concord
Boston
Albany
Buffalo
Erie
Toledo
Cleveland
Columbus
Cincinnati
Louisville
Evansville
Vincennes
Terre Haute
Springfield
Peoria
Rock Island
Davenport
Dubuque
Des Moines
La Salle
Madison
Milwaukee
Chicago
Lansing
Detroit
Toronto
Lake Superior
Lake Michigan
Lake Huron
Lake Erie
Lake Ontario
Mississippi R.
Missouri R.
Ohio R.
Red R.
Rio Grande
Tennessee R.
Cumberland R.
Kanawha R.
Monongahela R.
Allegheny R.
Hudson R.
Delaware R.
Susquehanna R.
Potomac R.
James R.
Roanoke R.
Yadkin R.
Pee Dee R.
Santee R.
Savannah R.
Ogeechee R.
Altamaha R.
St. Johns R.
Apalachicola R.
Alabama R.
Tombigbee R.
Pearl R.
Yazoo R.
Arkansas R.
White R.
Illinois R.
Wabash R.
Kentucky R.
Green R.
Licking R.
Muskingum R.
Hocking R.
Scioto R.
Great Miami R.
Little Miami R.
Wisconsin R.
Minnesota R.
Iowa R.
Kansas R.
Platte R.
Yellowstone R.
Missouri R.
Big Sioux R.
Jefferson City
St. Joseph
Kansas City
Omaha
Lincoln
Sioux City
Pierre
Bismarck
Fargo
Duluth
Minneapolis
St. Paul
Madison
Milwaukee
Chicago
Lansing
Detroit
Toledo
Cleveland
Buffalo
Albany
Boston
Portland
Augusta
Montreal
Quebec
Ottawa
Toronto
Lake Superior
Lake Michigan
Lake Huron
Lake Erie
Lake Ontario
NORTH DAKOTA
SOUTH DAKOTA
MINNESOTA
WISCONSIN
MICHIGAN
ONTARIO
QUEBEC
MAINE
VERMONT
N. HAMP.
MASS.
CONN.
R. I.
NEW YORK
PENNSYLVANIA
NEW JERSEY
DEL.
MD.
VIRGINIA
WEST VIRGINIA
OHIO
INDIANA
ILLINOIS
IOWA
NEBRASKA
KANSAS
MISSOURI
KENTUCKY
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
GEORGIA
ALABAMA
MISSISSIPPI
LOUISIANA
ARKANSAS
TEXAS
INDIAN TERR.
100 75 50 25 0 100 200

final destination. How much will be saved by this method of shipment is not stated."

It is thought that the tolls charged on the Panama Canal should be low—not much more than enough to cover the operating and maintenance expenses. The construction cost should be charged largely to National protection, at least until a large traffic is developed.

The map, Plate CXXVIII, accompanying a report of an examination of the Ohio River made by a Board of Engineers, United States Army, and transmitted to Congress in January, 1908, shows the territory that it is thought will be commercially benefited by reliable navigation in the Ohio-Mississippi system.

The Report states:*

"The boundary line of this territory was found by drawing a line through towns to which the freight rate from New Orleans by river and rail at present rates is equal to the present all-rail rates from seaport towns to the same points. Barge freight rate from New Orleans to distributing points such as Vicksburg, Cairo, Evansville, Louisville, Cincinnati, Wheeling and Pittsburg was assumed at 1.14 mills per ton-mile, which was the rate charged during the year 1905-6. The rail rate used was 4.832 mills per ton-mile from Gulf cities northward and is the present mean ton-mile rate on sugar in carload lots from New Orleans to Chicago, Cincinnati, Louisville, and Pittsburg. The rate used from Atlantic Coast cities inland is that of the Pennsylvania Railroad from New York to Pittsburg, or 7.4 mills per ton-mile, which is the mean of the ton-mile rate on sugar and rice. A rate of 2 mills per ton-mile was used on the Hudson River and through the Erie Canal, and a rate of 1½ mills per ton-mile on the Great Lakes for such commodities as sugar, rice, etc.

"Within the territory shown on the map now lives a population of about 20 000 000 people, 8 504 000 of whom live in that portion of territory lying in the Ohio Valley proper."

Methods of Improvement.

The original method of improvement of the Ohio River included the "removal of snags and rocks; the closing of duplicate channels by low dams; the guiding of the water into a single low-water channel by dykes where the river had an excessive width; and the removal, by dredging, of hard bars and projecting points."

It was only hoped by this method to prolong the navigation season for packet business, enough water not being available for producing

* Executive Doc. 492, 60th Cong., 1st Sess., p. 22.

a depth and width sufficient for towboats and fleets. Economy of transportation on the Ohio being within limits proportional to available depth and width, experience indicated that fleets from Pittsburg to New Orleans should draw at least 9 ft., and that channel widths of at least 300 ft. were necessary at the lowest navigable stage. This navigable condition could only be brought about permanently by materially increasing the low-water flow of the river, or by locks and dams.

The following methods of increasing the low-water discharge have been proposed:

(1).—To obtain the additional water needed during the rainy season by cutting a ditch from Lake Erie to the Ohio River. However, as the river above Marietta, Ohio, was at a higher elevation than Lake Erie, this method was impossible.

(2).—That Lake Chautauqua should be used as a regulating reservoir for the Ohio River. The drainage area of the lake was too small, however, for serious consideration. It was then proposed to use Lake Chautauqua as a reservoir, and to keep it filled by pumping from Lake Erie.

(3).—Colonel Charles Ellet, a civil engineer engaged in work at Wheeling, West Va., made some observations of the discharge of the river there, from which he reached the conclusion that the average annual discharge of the river was sufficient to maintain a continuous 6-ft. navigation, if the water was stored in reservoirs at times of large discharge and permitted to flow at times of small natural discharge. The plan proposed involved the construction of large dams to form reservoirs upon the head-waters. This scheme was reported upon adversely by W. Milnor Roberts, United States Civil Engineer, who was in charge of the Ohio River improvements from 1866 to 1870. Mr. Roberts was peculiarly well qualified to investigate the subject, both because of his efficiency as a civil engineer, and also because of his intimate knowledge of the region drained by the Allegheny and Monongahela Rivers, he having been connected at different times with the Pennsylvania State Canal between Johnstown and Pittsburg; the canal between Erie and Beaver; the Monongahela slack-water system; and surveys for various railroad lines in the regions mentioned.

This plan has recently been revived. It should be remembered, in considering it, that an open-channel depth of at least 11 ft. is now necessary, the trend being always for deeper water. A 14-ft. depth

is being advocated between Chicago and St. Louis. The records seem to indicate that the supply of water is not sufficient to provide such a depth during the entire year in the Upper Ohio River, while there may be sufficient for the Lower Ohio.

The proposition to increase the low-water flow by means of water impounded in reservoirs, had, of course, the double object of thus improving navigation and, at the same time, providing for flood prevention. The extreme irregularity with which floods occur and the navigation necessity of always having full reservoirs at the beginning of the dry season, would result many times in the reservoirs being full when needed for flood prevention. Thus they would not be able to restrain the flood in such manner as to prevent entirely the usual damage.

If flood prevention alone be the function of the reservoirs in the Upper Ohio, the writer believes that they could be made to serve such purpose. During the winter season, when the movable dams were down, the impounded water might be used to prevent the formation of ice in large quantities, by increasing the flow so as gradually to drive out such ice as forms, thus acting as an auxiliary to the present movable dam system. The water of the Chagres River, on the Isthmus of Panama, is to be impounded, in order to control its flow, in a reservoir which will be 165 sq. miles in area. The flood discharge of this river is about one-third that of the Ohio at Pittsburg.

Reservoirs as means of river improvement have always been attractive. Their cost, however, in connection with the danger of their being filled with detritus from the surrounding hills, has prevented their adoption generally as a system of river improvement, both in the United States and in Europe, and that on streams where the slope was such that commerce could obtain the full benefit, if the necessary depth existed.

A company was organized in 1855 for the purpose of experimenting on the Ohio River with a system of improvement which had been devised by General Herman Haupt. The Company proposed to prosecute the work at its own expense and on condition that no payment should be made by the Government unless the plan proved to be a success. In brief, the plan of improvement involved the use of low dams and open chutes, about 300 ft. wide, inclosed on one side by the natural bank of the river and on the other by a longitudinal bank or

mound. This plan was reported upon adversely in 1870 by Mr. Roberts, and again in 1880 by a Board of Engineers.

Mr. Alonzo Livermore proposed that low dams be used, but that, instead of locks, long chutes should be provided. These chutes would contain division walls and enlargements located so as to decrease the velocity and the discharge of water through the chutes.

Mr. Roberts investigated all the above-mentioned plans and then decided to recommend that the river should be canalized, locks and fixed dams being used, each dam being provided with a navigable chute for use during freshets of moderate height.

A few years later, the late W. E. Merrill, M. Am. Soc. C. E., Major, Corps of Engineers, United States Army, investigated the subject of a radical improvement of the river and agreed with Mr. Roberts that a lock and dam system was the proper solution of the problem. He stated that the advantages of a canalization scheme over any other proposed method of improvement were as follows:

(1).—It has been long tried and is now in use on the Monongahela River, where it meets the demands of the same commerce that navigates the Ohio.

(2).—There are no great hazards connected with the system, since the dams are low and the destruction of one would not necessarily injure the one next below.

(3).—It is known positively that locks and dams can be built, that will answer the purpose fully, and their cost can be determined beforehand with very fair accuracy.

(4).—There would be no damages from overflow, or destruction of property of any kind.

(5).—No special care is needed in the use of the slack-water system. The pools are themselves reservoirs containing the minimum quantity of water needed, and at the exact place where it is to be used.

(6).—The cost of the system would probably be less than that of the other (reservoir) system.

(7).—The pools would make excellent harbors for all river craft, an improvement that is greatly needed at the large cities and especially at Pittsburg.

There was one great objection, however, to the construction of any system of dams such as had ever previously been built in the United States. At times of freshets, all the coal shipments were made in fleets

which were too large to go through a lock of any reasonable dimensions, without being divided into two or more lockages. When the fleets are made up, the barges are fastened together very securely by cables in every direction, and the delays which would be caused by making and unmaking a fleet at each lock would be very great. (Plate CXXVII.) The coal shippers were vigorously opposed to the construction of any system of dams which would not permit the passage of fleets down stream, at times of freshets, without being broken up and reformed at each dam. It was believed that the influence of these interests was such that they would be able to prevent the appropriation by Congress of funds for any system of dams which would change the established system of coal carriage during high stages of water.

A Board, consisting of Majors Weitzel and Merrill, then made a careful study of the whole subject. A system of fixed dams was studied first, each dam to have an opening 4 ft. deep and 200 ft. wide, cut in the top with an inclined plane below the dam, and built so that, at high stages, coal fleets could be passed through the opening and down the inclined plane into the lower pool. The opening in the dam was to be closed by some form of movable gate which could be operated easily and quickly.

The Board studied the French and other systems of movable dams, but considered that there might be objections to the use of Chanoine wickets on the Upper Ohio, which did not exist in the case of the rivers of France. It was believed that the large quantity of drift and ice found in the Upper Ohio might be very injurious to the parts of a movable dam of the French type. Major Merrill's plan was finally adopted in part. This plan provided for 13 locks and dams between Pittsburg and Wheeling, built for a minimum depth of 6 ft., the dams to be of the movable type, with navigable passes 400 ft. wide, to be closed by Chanoine wickets, and high and low weirs, to be closed with either Desfontaines' wickets or Brunot's gates; the locks to be 78 ft. wide by about 630 ft. long, with lifts of from 4 to 7 ft. The first dam was located at Davis Island, five miles below Pittsburg. The plans for the first lock and dam were subsequently modified so that the lock as built was 110 ft. wide and 600 ft. long, and the weirs were closed by Chanoine wickets of shorter lengths than those of the pass. All wickets were originally planned to be maneuvered from service bridges. Davis Island Dam was commenced in 1878, and opened to navigation in 1885, and its cost was about $910 000.

The operation of the dam at Davis Island having been successful, in 1888, Major Merrill proposed the immediate construction of two more dams of the series. The Board of Engineers went still further and recommended the extension of the slack-water system as far as the mouth of the Beaver River, by the construction of four of the dams previously mentioned in Major Merrill's project of 1874. The first appropriation for building Dam No. 6 was made in 1890 and the last in 1902, and the lock and dam were completed and put in commission in the summer of 1904.

The first appropriation for the construction of Locks and Dams Nos. 2, 3, 4, and 5, was made by Congress in June, 1896, and the last in 1905. These latter dams were completed in 1906 and 1907. The time taken by Congress to appropriate the money for these dams is specially mentioned, because engineers are often blamed for the slow building of structures for which legislative bodies have not provided the funds, for which latter action engineers are in no way responsible.

In the construction of such a system of improvement as described in this paper, the building of each lock and dam complete should be provided for by one contract. This would prevent the delays due to entering into many contracts at different periods of the construction, and those due to collecting and erecting different sets of contractor's plant, etc.

Each lock and dam complete should be built in from 3 to 5 years, and there is no limit, of an engineering nature, to the number of locks and dams that can be contracted for at the same time. Of course, this means that all the funds necessary to complete a lock and dam must be authorized prior to the commencement of the work.

The River and Harbor Bill passed in the spring of 1905, authorized changes to be made in Locks and Dams Nos. 1 to 6, so as to provide for a navigable depth of 9 ft., instead of 6 ft., as originally planned.

The harbor of Pittsburg had been dredged so as to permit navigation by boats drawing between 9 and 10 ft., and, therefore, it was only necessary to change the plans of the dams which had not been completed, viz., Nos. 2, 3, 4, and 5. The changes made comprised the raising of these four dams by means of longer wickets and higher bear-trap gates, and the lowering of the lower lock sill at Dam No. 5. No changes had to be made in the completed dams, Nos. 1 and 6, but,

in order to permit of the extension of the harbor of Pittsburg to include a good natural deep-water pool a short distance below Dam No. 6, the lower lock sill at that dam was lowered. This will allow coal boats to be taken from Pittsburg during the navigable season, at any stage of the water, down as far as this good natural harbor, there to await a coal-boat freshet.

The Act of Congress, approved March 3d, 1905, made provisions for a survey of the Ohio River from Pittsburg to Cairo, with a view to the radical improvement of the river throughout its entire length. The report of the Board of Officers appointed to make the survey has been submitted to Congress. This report recommends the canalization of the river from Pittsburg to Cairo, so as to provide a navigable depth of 9 ft.; the locks and dams are to be built of practically the same type as those already constructed in the upper portion of the river; the Louisville and Portland Canal is to be widened, and a duplicate lock is to be placed therein. The estimated cost of the entire project was $63 731 488, in addition to the appropriation already made, viz., $9 281 376.

The locks recommended are to be 110 ft. wide and 600 ft. long in the clear. The dam is to consist of a navigable pass from 600 to 700 ft. wide, an automatic part, probably of the bear-trap type, and a series of movable weirs of the Chanoine, or other well-tried type; the proportion of weir and automatic part is to be determined by the conditions existing at each dam.

Thus far the order of building has been first to construct locks and dams below the important cities and tributaries, so as to provide good harbors as soon as possible with the funds available, and thereby enable river commerce to connect with existing railroads. Thus, Davis Island Dam was built below Pittsburg; and, at present, Dam No. 13, below Wheeling, No. 18, below Marietta, and No. 37, below Cincinnati, are under construction.

The following rules were promulgated by W. H. Bixby, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., for the location of locks and dams in the upper river:

"1st. To allow room in the river for a navigable pass at least 600 feet wide, with its centre nearly in line with the existing channel at tow-boat stages of water.

"2nd. To allow room in the river for weirs of at least 240 feet total length, so placed as to be as much as possible protected from injury by floating drift.

"3rd. To allow room in the river for passage of coal tows around each half of the dam during construction of the other half.

"4th. To allow of the sill or bottom of the navigable pass being placed at least slightly above the present bed of the river, and yet at least as far below the water surface as are the channel bars, which at coal-boat stages determine the available depth of the river in the pools above and below.

"5th. To avoid positions where the coal-tows must be flanked or placed obliquely to the channel line by reason of river bends, or oblique currents.

"6th. To allow of the view of dams by approaching boats for distances of at least half a mile both up and down stream.

"7th. To avoid positions near shifting sand or gravel bars, and mouths of sand-bearing rivers, creeks, and runs, unless such bars and streams be below the dams on the weir side.

"8th. To allow of positions below the mouths of large tributaries like the Little Kanawha, Great Kanawha, Guyandotte, Big Sandy, Scioto, Licking, and Big Miami, near enough to extend navigation properly to the nearest dam in such tributary, but yet far enough below the mouth of such tributary to avoid trouble from deposits and flood overflows, and to allow the proper handling of boats entering or leaving such tributaries.

"9th. To avoid cutting up or otherwise injuring the existing harbors of large cities, especially Parkersburg, Pomeroy, Point Pleasant, Gallipolis, Huntington, Catlettsburg, Ashland, Ironton, Portsmouth, Cincinnati, Covington, and Newport.

"The first, second and third conditions prevent locations in narrow parts of the river, and especially at islands where the bank channel is bare at low water, and of insufficient depth at coal-boat stages. The third condition prevents location within a half mile of the ends of islands, or high mid-river bars. The fifth and sixth conditions prevent locations at sharp bends.

"While it is not to be expected that all of these conditions can be fulfilled at each location, still past experience on this river, combined with the information so far secured, indicates that these conditions can, in general, be complied with, and that departures from the same will be the exception rather than the rule."

It is important, in planning any system of slack-water navigation, to foresee and provide for any increased depth which may be demanded in the near future. It is better to design the locks so as to accommodate a greater draft than that indicated at the time of construction, in order that increased depth may be easily obtained in the future by dredging in the pools. The extra cost of a lock so built is limited to the excess cost of the lower gate over the decreased cost of the lower

sill foundations. For this reason, any system of locks and dams should be designed so as to provide the draft needed, without the use of auxiliary dredging, leaving the deepening of the pools to be done later, to provide for a possible demand for increased depth.

It will be remembered that the dams originally proposed for the Ohio were fixed dams provided with a navigable chute. The requirements of the coal-fleet navigation made fixed dams which were not provided with some kind of navigable pass out of the question.

A movable dam to be used on the Ohio River should conform to the following:

It should be easily operated and free from complicated mechanism.

It should be capable of being lowered rapidly, without chance of failure.

Its parts should be so strong and simple as to prevent damage by ice and drift. This consideration is of very great importance on the Ohio, where the severity of the winters must be kept in mind in selecting a type of movable dam.

It should be capable of being easily operated so as to pass small freshets without the necessity of lowering the entire dam and without lowering the pool below normal level.

It should afford a navigable channel at least 600 ft. wide, with the dam down, and the passage should then have a depth of water equal to that on the bars above and below the dam. It is thought that the combination of Chanoine pass and weirs, with bear-trap automatic portion, fills these conditions better than any other.

The lift of the dams planned, up to the present, has been limited to about 8 ft. as a maximum. This was originally based on the experience in France. With these small lifts and a navigable stage of only 6 ft., the wickets for the navigable pass were of a length not greater than 14 ft. The increase in navigable depth on the Ohio, from 6 to 9 ft., caused the use of wickets as long as 18 ft. These have been maneuvered satisfactorily, and it is possible that Chanoine dams of a 10-ft. lift could be satisfactorily operated.

A wicket 16 ft. long was adopted in the project last recommended to Congress. If the construction engineers can safely lengthen these wickets, a smaller number of dams will be needed.

There are no lift walls in the locks, both gate-sills being at approximately the same elevation as the sill of the pass. This is advan-

tageous, because it permits boats to go through the lock at times when the dam is down and there is not much water in the river. When an upward-bound packet meets a tow going down stream through the pass, it is often desirable for the packet to pass up stream through the lock.

There are no guard walls prolonging the river wall of locks, because it has not been believed that they are necessary.

Where the foundations of the river walls of the lock are such that no serious scour may be anticipated on the outside, due to the operation of the dam, it is customary to let the dam start from about the center of the lock wall, thus affording ample room for emptying and filling the valves, which are placed in the outside wall itself.

Whenever there is danger of scour at the place designated above, it is customary to let the dams abut against the wall as near the lower end of the locks as possible, and, at the same time, to provide the necessary filling and emptying arrangements. Eddy currents below movable dams should not be very serious, since the dams are down at all higher stages of the river.

Before a sudden rise, it may be essential to commence lowering the navigable pass at the end next to the lock, although this should not be done unless absolutely necessary. If it is done, the scouring effect of the current alongside the lower end of the river wall is very great. It should not be concluded that, because movable dams are down during the higher stages, scour is not probable.

In case of accident to the lock gates, the rush of water through the lock would probably undermine the walls, unless they were properly protected against scour. Moreover, during construction, there are times when the cross-section of the river at the site of the dam is very much reduced by coffer-dams, etc., and the currents through the openings between obstructions are such as to produce very dangerous scour. The wall foundations should be protected against any of these contingencies.

At Lock No. 4, during construction, the entire width of the river was at one time closed, except about 430 ft. which was divided into two channels. One of these channels, next to the lock wall, was only 220 ft. wide. The scour along the lower end of the river wall, caused by the severe currents through this narrow passageway, threatened to undermine the lower end of the wall, and the scour was only stopped by the prompt placing of a large quantity of rip-rap, thrown by hand from the top of the wall.

The principle which should govern the height of the walls above the pools is that they should be of such a height as will permit a boat to lie alongside them with its guard below the top of the wall. This would indicate that 5 ft. above the water is the proper height to accommodate packet-boats, the guards of which are generally from 4 to 4½ ft. above the water surface, but, of course, this depends on the load carried. Towboats have much lower guards. As the lock may be used until the upper pool rises to a height of about 1 ft. above the crest of the dam, it would be better to build the upper guide wall and the land wall of the lock to an elevation of about 6 ft. above the level of the normal upper pool. The river wall need not be built so high, because it is unusual for packets to lie alongside that wall while making a lockage, and, if necessary, their fenders can be used at any time. In fact, the dams in the stretch of river near Wheeling have river walls of less elevation that the land walls; but, as originally built, the walls of the locks in the upper part of the river were uniformly 5 ft. above normal pool level. The increase in pool elevations, caused by the change from a 6- to a 9-ft. navigable depth, decreased the height of the walls above the pools and necessitated raising some of the walls. In most cases, the land wall and lower guide wall were raised, the river walls not being raised. The lower guide walls were originally built to an elevation of 5 ft. above normal lower pools, which provided the same clearance as the other walls, when the pools were at normal level. However, this did not take into consideration that when there is considerable flow over the dam, the lower pool rises almost twice as fast as the upper pool. It would seem then that for a symmetrical design, the lower guide walls should be built to an elevation of about 7 ft. above normal lower pool, if the land wall is to be 6 ft. above normal upper pool. A guide wall should be at least as long as a tow that can pass through the lock in one lockage. In the latest dams on this river, the guide walls are from 600 to 700 ft. long.

Lock-Gates.

The Ohio River locks are differentiated from most other locks in the United States principally by the type of lock-gate used. The gates are of the rolling type designed by the late Major W. E. Merrill. When out of use, a gate of this type is housed in a recess in the bank; when needed, it is run across the lock, thus closing it, the screen serv-

ing as a gate. As originally planned the locks were to be 78 ft. wide. It was not believed to be practicable to make them wider, because of the small height and the great length of the miter-gates. However, when it was deemed necessary, later, to increase the width to 110 ft., a form of gate suited to such a wide span and small height was designed. These rolling gates run on tracks in much the same way as an ordinary railway car. The principal parts of a gate of this type are:

First.—The top truss which serves to carry a portion of the water pressure to the lock walls.

Second.—The vertical water-screen which serves to close the lock and transfers one-third of the pressure to the top truss and two-thirds to the track.

Third.—The trucks, wheels, etc.

The original gates at the Davis Island Lock were made of pine timber. The top truss was of the Howe type, supported on posts so as to lie above the normal level of the upper pool. The posts extended below the level of the track, and their lower ends served as flanges for the wheels which were of the plain-tread type, traveling on a flat-rail track, the gauge of which was 11 ft. 6 in. The spacing of the rows of posts was such as to provide for a 2-ft. lateral movement of the gate; for the gate to move laterally, however, it was necessary for it to slide on the wheels parallel to the axis of the lock. This sliding created great strains on the lower part of the gate, and resulted in the breaking of many wheels and axles. The gate was moved back and forth by two chains, each chain being fastened at one end to an end of the gate and wound around a chain drum at the entrance to the recess. The chains were rigidly attached to the gate, and this mode of fastening was found to give much trouble. If the gate struck a submerged snag or rock on the track, it stopped moving instantly, and something broke before the engine could be stopped. Wheel axles were broken several times, and, on one occasion, when the gate could not be moved out of the recess, it was found that the outer axle was broken, the lower wheel of the second axle off, and the axle out of its bearings.

After the original wooden gates had served for 12 years they were replaced by steel gates. In these new gates the top truss is of the Pratt type, the wheels are flanged, and a lateral movement of the gate is provided for by hanging the framework upon each axle by eye-bar hangers, which allow the frame to swing laterally like a pendulum. This ar-

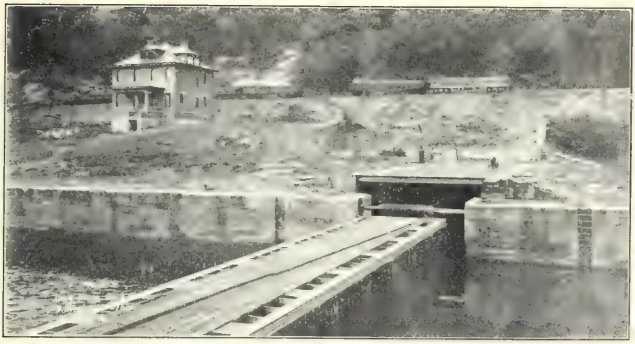


FIG. 1.—LOCK NO. 6, OHIO RIVER.

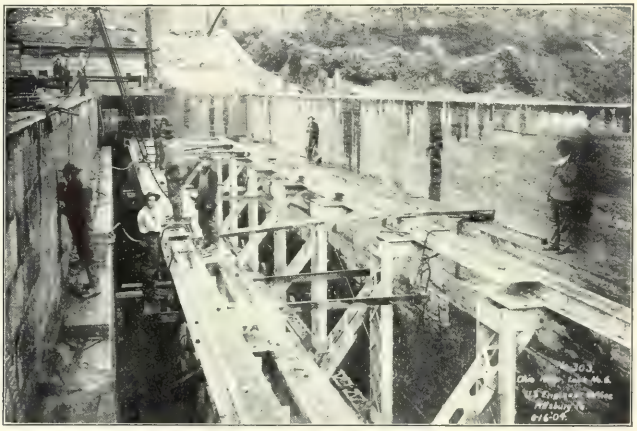


FIG. 2.—GATE RECESS, LOCK NO. 6, OHIO RIVER.

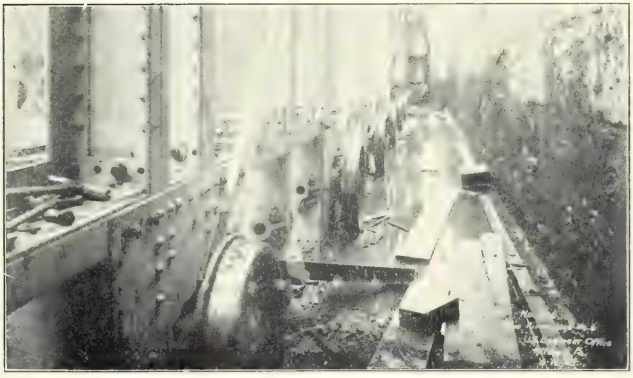


FIG. 3.—GATE FOR LOCK NO. 6, OHIO RIVER.

rangement prevents the friction of the bottom bearing from interfering with the opening of the gate. When the pressure is on the gate, it swings down stream about 2 in. until the bottom-bearing piece rests against the up-stream side of the down-stream rail. When the pressure is released, the gate swings back so that the bearing strip is about 2 in. away from the rail. In the new gate, rectangular valves are used instead of the circular butterfly valves of the original gates.

The latest gates are those at Locks Nos. 2, 3, 4, 5, and 6. They are of steel, and are somewhat similar to the steel gates at the Davis Island Lock. The top truss is also of the Pratt type. The water-screen consists of horizontal white oak planks bolted to vertical, 15-in., 42-lb. I-beams, which rest, at the top and at the bottom, against horizontal members riveted at each end to the down-stream posts of the gate. Below the intermediate horizontal member and the bottom water-coal strip, are the gate valves. There are two of these valves in each panel, making eighteen in each gate. The valves are rectangular, of the horizontal-axis, butterfly type, approximately 3 by 4 ft. in size. They are made of structural steel closing against oak cushions, and are operated by hand from the top of the gate by racks and pinions. The valves were not constructed to be operated by power, because the locks were provided with filling and emptying valves in the river wall, and the gate-valves are not to be used except in case of accident to the wall-valves, and possibly to assist in flushing deposit out of the lock chamber and the tail-bay. (Plate CXXIX.)

The pendulum arrangement for the lateral swing of the gate is much the same as in the later Davis Island gate already described. The connection between the superstructure of the gate and the trucks is through standard car springs which serve to prevent shocks to the gate when the wheels run over obstructions on the track. The wheels are standard car wheels, slightly modified in the tread so as to fit the level gate-tracks. The water-seal at the bottom is formed by a wooden bearing strip. The water-seal at each end of the gate is formed by a pivoted pipe arranged so as to be swung by the pressure of the water until it closes the opening between gate and wall. The operating chains are $1\frac{1}{4}$-in. crane chains fastened at each end of the gate to a tug-lever, which is arranged with spring control so as to diminish the shock due to starting and stopping the gate. In order to prevent breakage of the

chain, the connections to the tug-levers are made by shackles and pins of such a size that the pins will break before any link in the chain can possibly fail. The gate is moved back into the recess and out across the lock by the rotation of the chain drum at the entrance to the recess. This drum and its shaft are rotated through a system of gearing by a 10 by 12-in. engine of the horizontal, double-cylinder, reversible type, operated by compressed air, at Locks Nos. 2 to 6. At Davis Island a 12 by 16-in. steam-driven engine is used.

In the design of these gates, no estimate was made of the stresses which would result in case a gate should be struck by a boat entering the lock. All truss and screen members were designed to support the static water load, under the assumption that the lower pool had been drawn down to sill level. Safe working unit stress for steel members was assumed at 10 000 lb. per sq. in.

The collection of drift in the gate recesses and in the gates, has given great trouble at Davis Island. This is due largely to the fact that that lock is filled partly by valves in the upper recess, and is emptied partly by valves in the lower recess. The operation of these valves creates currents toward the recesses, which draw into them the drift accumulated in the forebay and in the lock-chamber. It is not believed that as much difficulty will be encountered at the other locks, as they are filled and emptied entirely through valves in the river wall. However, in order to prevent any drift from entering the recesses, it has been proposed that the gates built hereafter be provided with pervious screens on their up-stream sides, placed so that when the gate is moved, these screens will pass close to similar screens projecting from the up-stream sides of the recesses at the entrances. All drift can then be locked through, and none will be allowed to enter the recesses.

The gates at Locks Nos. 2 to 5 can be opened or closed in from 1½ to 2 min. The average time of operation at Lock No. 6, which has been in use for three seasons, is 2 min. The gates at Lock No. 6 have been moved while under a 6-in. head, the lower gate being closed while the filling valves were open and while there was considerable current in the lock-chamber.

The recesses in which the gates rest when the dam is down, and from which they emerge when the lock is put into operation, are 125 ft. long and 20 ft. wide in the newest locks under consideration. The recesses of the lock at Davis Island are so narrow that there is only

about 1 ft. clearance on each side of the gate. This has often proved to be too small to permit of making the repairs to the gates, which are constantly required. On one occasion it was necessary to raise the gate 2 or 3 ft. off the track in order to make repairs. When a gate is erected in place on the tracks, which is the usual method, considerable room is needed during the erection. Moreover, if the gate be erected on the track in the recess after the lock-chamber has been flooded, a sump should be provided inside the entrance to the recess, and there should be room for the necessary suction pipes, etc. It is often desirable to use a horizontal, centrifugal pump in unwatering the recess, and if such a pump is not placed on some kind of floating support, it is desirable that it be placed in the recess. It is well, therefore, to make the recess large enough to permit this arrangement. It is believed that the recesses should be provided with vertical slots in the side-walls near the entrance so that horizontal timbers can be placed in the slots, thus forming the sides of a coffer-dam.

The gate-recesses at Locks Nos. 2 to 6 are wider, but not longer, than those at Davis Island. It is believed that a recess should be about 135 ft. long for a lock 110 ft. wide, the gate being about 118 ft. long. It has been noted that the gauge of the gate tracks is 11 ft. 6 in. They were so constructed, but, just prior to the erection of the gates, at Locks Nos. 2, 4, and 5, it was found that the gauge of the track at the entrance of the recesses was from 1 to 2 in. too small. Inasmuch as no greater clearance than $\frac{3}{4}$ in. had been provided in the gauge of the gate tracks, it was necessary to tear up and reset a portion of the track. It is believed that expansion of the concrete in the long land walls and guide walls was responsible for the narrowing of the track. To prevent any movement of the track rails, it has been proposed to use ties made of two channels, back to back, with the rails riveted to an interior gusset plate at each end of the tie, these steel ties to be used instead of the former wooden ties.

There having been considerable trouble at the Davis Island Lock, caused by the collection of drift in the gate recesses, drift chutes were built when the guide walls were rebuilt of concrete. In Locks Nos. 2 to 6, the drift chute is a tunnel running from the rear of the upper gate recess down past the rear of the lower gate recess (with which it is connected) and emptying into the tail-bay of the lock. The entrances to the drift chutes are provided with movable doors, and are intended for

flushing the drift out of the recesses. The construction of the drift chutes is very expensive, especially so when rock is found at a high elevation. It seems that it might be well to omit them and solve the drift problem by collecting the drift below the mouths of the tributaries, and not be bothered with the same drift at every dam on the river (Fig. 1). Of course, the greatest quantity of drift is running at times of freshets, when the dams are down, and the only drift to be cared for by the method proposed would be that which is floating on the pools. This will probably decrease in quantity.

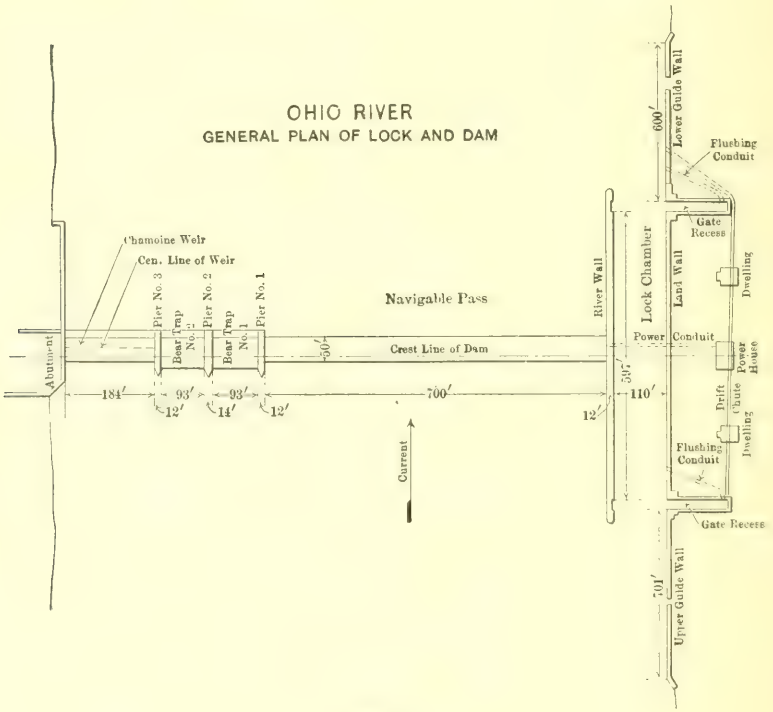


FIG. 1.

The flushing conduits are small tunnels which run from near the rear of the gate recesses and empty into the lock-chamber and tail-bay. They are intended to be used in flushing all sedimentary deposit out of the recesses. They are provided with Stoney gate-valves at the entrances, and if used when the corresponding gate is across the lock-chamber,

a head equal to the lift of the dam is available for flushing purposes. They will work very nicely if the deposit is so small in quantity as to permit the gate to be run across the lock-chamber, but if the gate can be run across the lock-chamber, why is any flushing necessary? It is noteworthy that, although flushing conduits were built at Lock No. 6, they have never been used, and the lock has been operated continuously for three seasons. After the high flood of March, 1907, the quantity of deposit in the gate recesses was very great. In some of them there was found a sedimentary deposit 7 ft. deep, and, from the two gate recesses at Lock No. 2, approximately 1 400 cu. yd. had to be removed. The deposit at all the locks after this flood was so great that, before any gate could be moved, it was necessary to coffer off the recess and remove the deposit. The flushing conduits were of no use because no head was available, the dams being down and the gates stuck fast. In order to ensure the satisfactory working of the lock-gates and to increase their durability, it is believed that it is wise to pump out the gate recesses every spring prior to the first raising of the dams. The débris can then be removed, the gates repaired, if necessary, and painted.

Gate recesses are provided with roofs built either of reinforced concrete or of I-beams supporting ½-in. steel plates. The last-mentioned type is believed to be the better, as it can be easily removed prior to making any needed repairs to the gates. The roofs are placed at such a height as will permit a man to walk on top of the gate and push the drift out of the recess, or into the drift chute.

Each lock is provided with a conduit which runs across under the lock from the bottom of a well in the power-house to the bottom of another well in the river wall. It is in this conduit that the pipes are laid which supply pressure for working the river-wall valves and the bear-trap gates. In placing or repairing these pipes, it is necessary to pump out the conduits, and they should be provided with a sump at the bottom of one of the wells. If used at all, these power conduits should be placed so low that they cannot be injured by any dredging that may be required in cleaning out the lock-chamber. In the later designs, the conduits have been omitted, the power pipes being placed in troughs in the gate-track foundations.

At Davis Island, the lock is filled through seven, circular, butterfly valves, 4½ in. in diameter, in the river wall, and the same number

in the down-stream wall of the upper gate recess. The lock is emptied by seven valves of the same kind and size in the down-stream wall of the lower gate recess, which all discharge into a large conduit emptying into the tail-bay. Gate-valves were also used. The wall-valves were provided with vertical axes, and were turned by lever arms moved by a large hydraulic jack, one jack turning seven valves. The valve stems had rigid connections with the hydraulic jacks. Because of obstructions getting in the valves, most of the valve stems became twisted and the valves would not close, thus making the leakage great. A spring compensating device was originated, which permitted any one valve to remain open while the others were closed. At Lock No. 6, considerable trouble was had with the breaking of springs in this compensating device, and individual jacks, arranged in a parallel closed circuit, were adopted for use at Locks Nos. 2, 3, 4, and 5, each valve being provided with one of these little engines. The jack consists of a cylinder of Shelby drawn-steel tubing, $5\frac{1}{2}$ in. in diameter, provided with cylinder heads, cross-head guides, etc. Motion is given to a lever arm by a connecting rod operated by a piston of 30-in. stroke. The parts were proportioned so that the valve shafts could not be twisted by any force which could be transmitted to the lever arm by the pistons. The shaft of any valve not operating would not be injured, therefore, while the others turned. The valve shafts are vertical, and made of cold-rolled steel, $3\frac{1}{4}$ in. in diameter.

The valves at all the locks, except those at Davis Island, were placed in the river wall of the lock—sixteen above the dam, for filling the lock-chamber, and sixteen below the dam, for emptying it. The valves are circular, butterfly valves, $4\frac{1}{2}$ in. in diameter, of cast iron, in a single piece, with horizontal stiffening ribs.

The individual jacks have been highly successful. They are operated by water, except in the coldest weather, when oil is used. The pressure is obtained from a duplex pump in the power-house. The piping is installed so that, while water under pressure is supplied at one end of the jacks from the common supply line operating the pistons and valves, the water on the other side of the pistons flows into the other supply line and back to the tank in the power-house. A four-way valve controls the flow, so that the direction of motion may be reversed.

FIG. 1.—MANEUVERING BOAT AT LOCK NO. 2, OHIO RIVER.

FIG. 2.—OPENING THE DAM AT LOCK NO. 2, OHIO RIVER.

Power-Plants.

At Davis Island, the steam for operating the lock-gate engines was generated in a boiler at the lower end of the locks; the steam for the upper gate engine was piped about 600 ft. to the engine; so much trouble was had from condensation that a boiler was installed at the upper recess. At Lock No. 6 the boilers were assembled in the power-house, which was opposite the middle of the lock, and steam was piped a distance of about 300 ft. to each gate engine; here again considerable trouble was encountered on account of condensation. A single-stage air compressor, with a capacity of 200 cu. ft. of free air per min., had been installed to assist in moving the bear-trap gates. This was replaced by a two-stage steam-driven compressor, with a capacity of 400 ft. per min., and larger storage tanks were installed. Thereafter the gate engines were operated more satisfactorily with compressed air.

The valve engines at Davis Island were originally operated by water pressure, supplied by elevated tanks under a head of about 62 ft. It proved to be hard to move the valves when under a head exceeding 5 ft., and later the pressure was obtained directly from steam pumps.

The plants at each of the recently completed locks consist of two Westinghouse, three-cylinder, vertical, gas engines, operating on natural gas, and two chain-driven, Blaisdell, two-stage air compressors, each having a free-air capacity of 500 cu. ft. per min., at a pressure of 100 lb. The operation of the bear-trap is facilitated by air forced into the lower leaves, direct from the air tanks. The gate engines are also operated by compressed air. The filling and emptying valves are operated by liquid pressure, as already stated, the pump being operated by compressed air. For maneuvering the gates, it is simply necessary to have a man at the throttle of the gate engines, but all operation of valves is from the power-house. Cooling water for the engines and compressors is obtained from elevated tanks supplied by an air-driven pump in the basement of the power-house.

One great advantage in operating the valve engines by liquid pressure is that, in case of accident to the supply of gas, a small boiler can be coupled quickly to the supply pipe of the operating pump, and the valves can be operated in the same way as when the pressure pump was air-driven. If the valve engines were operated directly by air pressure, this could not be done.

The Dam.

The dams are principally of the Chanoine wicket type. The part of the dam over which navigation passes when the dam is down is called the navigable pass. In these dams, the navigable pass is located next to the lock, so that entrance to the lock will not be disturbed by currents from the weirs, etc. Each dam contains regulating weirs of the bear-trap variety. They are separated from the abutment shore by a weir of the Chanoine type, except at Lock No. 6, where there is a weir of the A-frame type. The current through the bear-traps is so swift and the scour below them so great, that, if they are placed next to the bank, considerable extra expense must be incurred for bank protection.

In raising one of these dams, the pass is generally closed, first by the maneuvering boat; then the Chanoine weir is closed by a winch running on a service bridge, and the bear-trap, or automatic part of the dam, is closed last—the raising of the rest of the dam having created head enough to permit the raising of the bear-trap gates either by hydrostatic pressure alone, or assisted by air. (Plate CXXX.)

The requisite weir area is important. It should be sufficient to permit of closing the pass without causing a rise of water surface which will prevent the maneuvering boat from raising the last wickets, the difficulty being in catching hold of the wicket.

While the pass is being raised, discharge is taking place through the Chanoine weir, through the bear-traps, and through the part of the pass not yet raised, as well as through the spaces between the wickets already raised. In calculations, all these points must be considered. Considering now the lowering of the dam because of a rise in the river, the discharge area should be manipulated so as to pass the stream flow without the necessity of lowering all the dams until the discharge becomes large enough to maintain a natural stage of 9 ft. in the river. The upper pool may be allowed to rise until it is about 1 ft. above the crest of the dam, without preventing the lowering of the dam from a maneuvering boat.

It is important that the amount of automatic weir area be not made so great that the flow of the river through the openings can take place without creating head enough to operate the weirs.

In designing the foundations of the dam, the height of the sills or highest fixed parts must be first decided. This is governed by the

OHIO RIVER

CROSS-SECTION OF DAM NO.6

DRAWN UNDER DIRECTION OF MAJOR W.H. HEUER,
CORPS OF ENGR'S., U.S.A.
JULY 10TH, 1896.

OHIO RIVER

CROSS-SECTION OF FOUNDATION OF DAVIS ISLAND DAM (DAM NO.1)

DRAWN UNDER DIRECTION OF COL. W.E. MERRILL, CORPS OF ENGR'S., U.S.A.

DATE NOT KNOWN

desirability of not interfering with the normal flow of the river and also by the requirements of navigation. The sill of the pass should be placed at least as far below low water as the crests of the nearest bars above and below the dam. If the sill is placed too low, there will be unnecessary trouble in handling the longer wickets, and the foundation will cost more because of being further below the surface of the water. If the sill of the pass is placed too high, it may interfere with open-river navigation.

The sills of the weirs are placed at higher elevations than that of the pass. A Chanoine weir provided with a high sill will have short wickets, and can be easily operated so as to assist in passing small rises without lowering the entire dam, and thus lowering the pool.

The various types of foundations used in these dams are shown on Plates CXXXI, CXXXII, and Figs. 2 and 3.

The general considerations which should govern in designing a dam foundation are, that there should be a water-tight surface at the up-stream face, and that the remainder of the dam should be constructed so as to maintain the water-seal in place and prevent its disturbance by scour below the dam or underneath it. In all these dams the water-seal is practically the same. The dams differ in the style of support for the concrete, and in the character of the protection against scour below the dam. Below Dam No. 2, and below portions of Dams Nos. 3, 4, and 5, stone-filled apron cribs were used, heavy rip-rap being placed down stream from the cribs. Below Dam No. 6, and below part of Dam No. 4, protection against scour is afforded by a wide apron of rip-rap held in place by piles driven in quincunx order. This style is similar to that used in the dams on the River Yonne. Below a portion of Dam No. 5, a wide concrete apron, in which were embedded old wire ropes, was used.

The question of scour below dams is of great importance. It has been necessary at Davis Island on many occasions to place rip-rap below the dam, at one time whole barges loaded with stone being sunk as a protection. The scour is especially bad both above and below the bear-trap gates. Soon after Dam No. 6 was put into operation, scour developed to such an extent around the piers and bear-traps that it reached to the bottom of the concrete at the head of the piers, and just below the protection apron, the scour went to bed-rock, about 31 ft. below low water. It was necessary to place about 2 000 tons of rip-rap to stop this scour.

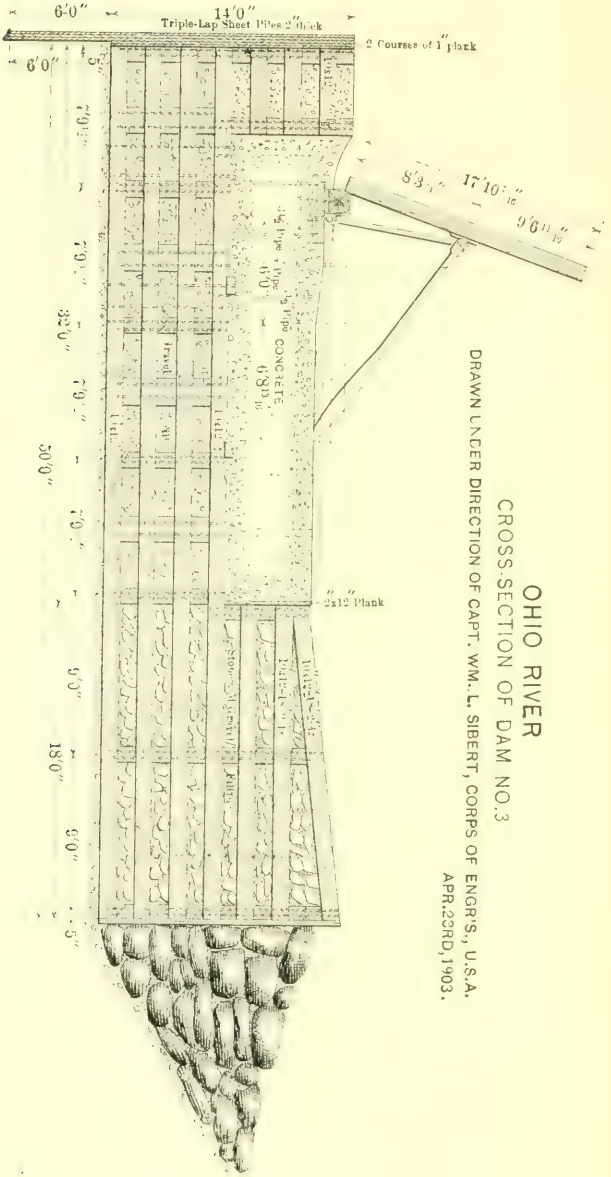

6'0"
14'0"
Triple-Lap Sheet Piles 2" thick
2 Courses of 1" Plank
6'0"
8'3"
17'10"
9'6"
CONCRETE
9'0"
6'8"
35'0"
50'0"
12"x12" Plank
9'0"
18'0"
9'0"
OHIO RIVER
CROSS-SECTION OF DAM NO.3
DRAWN UNDER DIRECTION OF CAPT. WM. L. SIBERT, CORPS OF ENGR'S., U.S.A.
APR.22RD,1903.

FIG. 2.

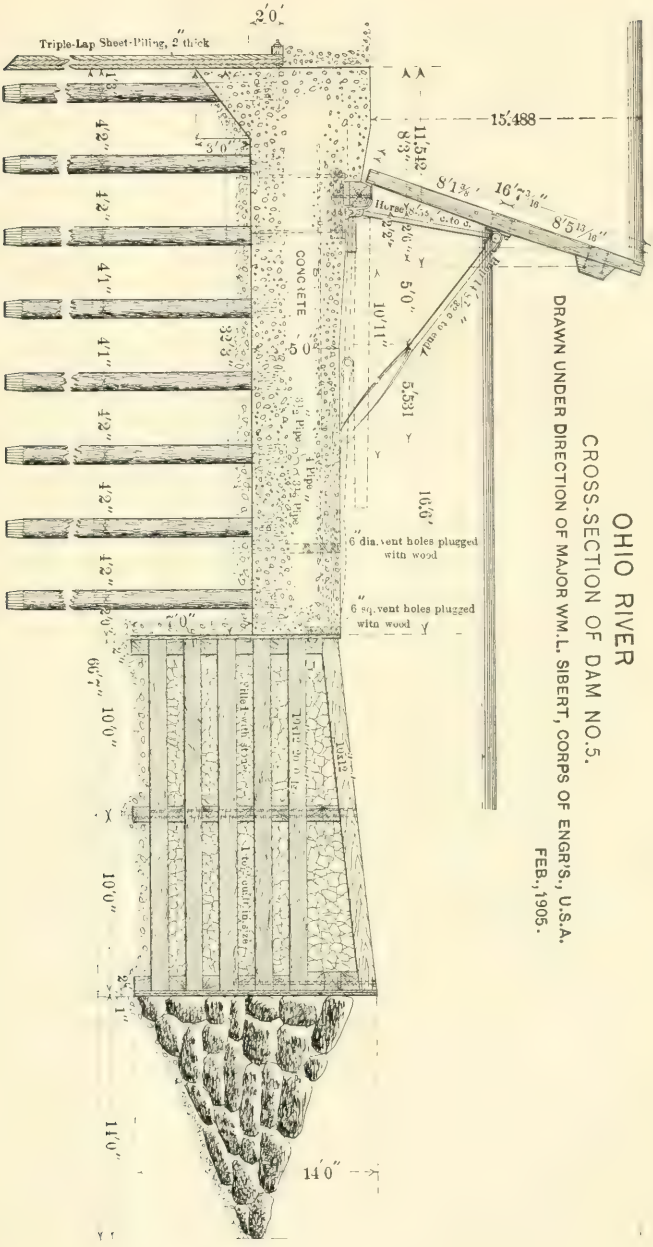
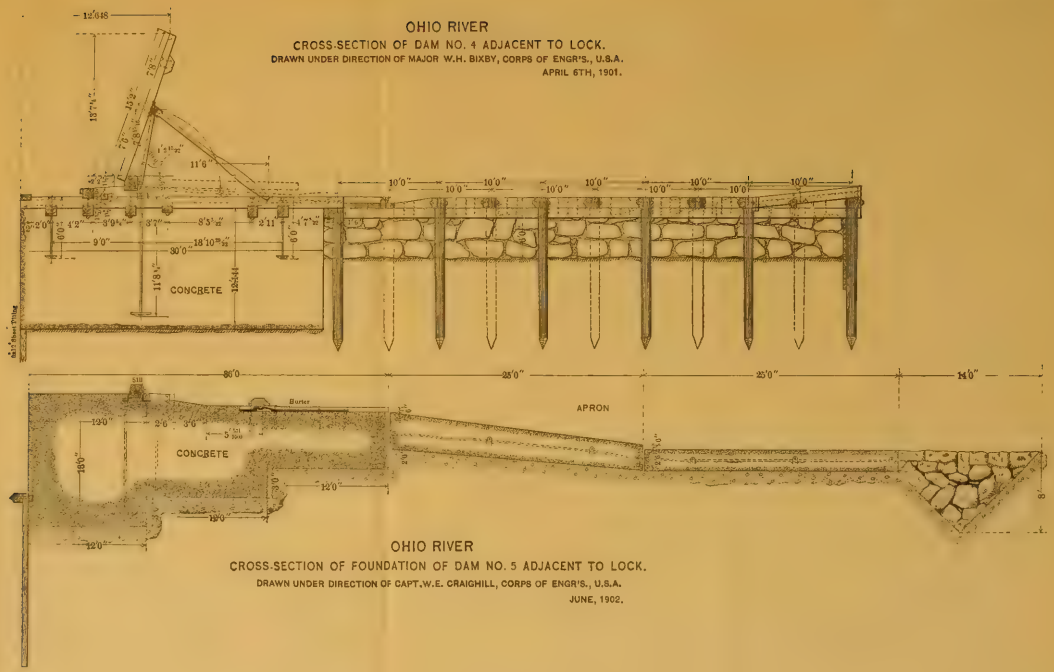

OHIO RIVER
CROSS-SECTION OF DAM NO.5.
DRAWN UNDER DIRECTION OF MAJOR WM. L. SIBERT, CORPS OF ENGR'S., U.S.A.
FEB., 1905.
2'0"
Triple-Lap Sheet-Piling, 2" thick
15'488
11'542
8'3"
8'1⅜"
16'7 3/16"
8'5 13/16"
Horses
2'0"
5'0"
10'11"
5'531
16'6"
CONCRETE
5'0"
6" dia. vent holes plugged with wood
6" sq. vent holes plugged with wood
1'0"
65'7"
10'0"
10'0"
2"
1"
11'0"
14'0"

Fig. 8.

The type of foundation, shown in Fig. 3, is that used, in the latest foundation work on the river, when rock at practicable elevations does not exist. The water-seal is formed by the wickets, by the up-stream edge of the concrete, and by a row of 9 by 12-in. Wakefield sheet-piles. The concrete is supported on round piles, driven to rock if possible. It is a general principle of construction that the upward pressure beneath a dam should be made as small as possible. This is done by making sure that the up-stream face of the dam is more nearly water-tight than the down-stream face. As there are bound to be leaks between the sheet-piles, an effort has been made in designing these dams to provide free egress for the leakage water in such a way as to insure no movement of the material under the dam. This is done by passageways leading up from underneath the tail of the concrete and opening into the cribs. In order to permit of inspection of the condition of the foundation beneath the dam, vertical holes, 6 in. in diameter, were left in the concrete near the down-stream edge, at intervals of 8 ft. from center to center. These were plugged with swelled wood. The plugs can be removed, or holes can be bored through them and soundings taken to ascertain the conditions existing underneath the dam. In case of scour, the opening can be filled with grout or concrete.

At some of the dams, a wooden covering was placed on top of the foundation concrete, the timbers being anchored to the concrete. The idea was to prevent damage to wickets by interposing an elastic cushion between them and the concrete. It will soon be necessary to repair the wooden top at Davis Island, and it will be a hard task. The wear at the top has been caused very largely by the props running out of the hurters and gouging holes in the wood. It might be advisable in designing the foundation for a movable dam of the Chanoine type to arrange the top surface so as to fit a movable coffer-dam which could be used in replacing movable parts, making minor repairs to the foundation, etc.

The wickets used in the Chanoine dams on the Ohio River are 3 ft. 9 in. wide, and the largest are approximately 18 ft. long. As they are placed 4 ft. apart from center to center, there are 3-in. spaces between the wickets, which serve to prevent interference of the wickets, due to an oblique pull in raising, etc. The discharge of the Ohio River is sufficient to permit of this arrangement, and when the discharge is very small, wooden needles or joint covers are used to cover the spaces



OHIO RIVER

CROSS-SECTION OF DAM NO. 4 ADJACENT TO LOCK.

DRAWN UNDER DIRECTION OF MAJOR W.H. BIXBY, CORPS OF ENGR'S., U.S.A.

APRIL 6TH, 1901.

OHIO RIVER

CROSS-SECTION OF FOUNDATION OF DAM NO. 5 ADJACENT TO LOCK.

DRAWN UNDER DIRECTION OF CAPT. W.E. CRAIGHILL, CORPS OF ENGR'S., U.S.A.

JUNE, 1902.

between the wickets. The wearing away of the wickets has sometimes made it impossible to stop all leakage at the Davis Island Dam, even when the needles are used, and, at times, straw and willows have been used to decrease the leakage. This pool, however, has been maintained for years with no pool below, when leakage was greater than it will be hereafter, this part of the system having been completed.

The pass wickets should be designed so that they will not swing, at least until the upper pool rises to a height of 2 ft. above the crest of the dam. The water has been allowed to rise that high at the Chanoine Dam in the Allegheny River; however, that was under exceptional circumstances. It is generally necessary to commence to lower the pass by the time the upper pool is 1 ft. above its normal level. In order to assist the wicket in maintaining its upright position, cast-iron counterweights are placed in the breach of the wicket. The desirability of making the pass wickets non-automatic until a certain stage is reached, puts an inferior limit on the distance from the foot of the wicket to the journal boxes, and the economy of keeping the length of the horse and prop small provides the superior limit.

It is customary, in the operation of a Chanoine dam in the Ohio, to leave one wicket of the pass down near the river wall of the lock. This affords a passage for small drift which collects at times near the lock wall.

The horses have a large factor of safety when the normal tension to which they are subjected is considered; but very often they have been damaged by being struck by boats, snags, etc. Those in the dams described generally have been provided with side bars $1\frac{1}{2}$ by 3 in. in section. The latest ones have 2 by 4-in. side bars. At first, the axles were made of cast steel, but many were broken, and forged steel axles were then adopted.

The props have been $3\frac{1}{2}$ to 4 in. in diameter, with a large excess of material at the lower end, in order to ensure that they will remain in position in the hurter channel when the dam is being raised. The eddy currents have a tendency to swing the props sidewise, and to prevent their seating properly against the step in the hurter. It has been considered by some that the large bulge at the bottom end of the prop contributes to this by offering a greater surface for the water to act against, and of late experiments have been made with cylindrical props, $3\frac{1}{2}$ in. in diameter throughout their length. It sometimes happens in

lowering the wickets that the props do not run back properly on the hurter grooves, but catch and prevent the wicket from falling. It is then customary to lift the lower end of the prop with a hook on the end of a line from the maneuvering-boat derrick, this hook engaging in a notch in the lower part of the prop.

The current increases greatly as the last few wickets are reached in raising the dam, and these wickets should be provided with heavier props than the other wickets of the pass. One of the greatest troubles encountered in raising the pass is the difficulty of catching hold of the handle plates of the last few wickets to be raised. It is believed that the wickets of the pass nearest to the first pier might well be provided with chains reaching from the bottom handle plate of one wicket to the top handle plate of the next wicket on the side toward the lock. The last few wickets can then be raised by these chains.

The Bear-Traps.

A bear-trap weir was installed at Davis Island in 1889. The gates were of wood, and were 52 ft. long, forming a weir of the type known as the "old bear-trap." It was intended for use as an automatic weir for the regulation of the pool levels, and also as a drift-pass. This bear-trap weir proved to be of much assistance in operating the dam. After the original bear-trap at Davis Island had been in service for about sixteen years, it was replaced by a new bear-trap, the leaves being of wood heavily bound with steel, and proportioned on the basis of 100 upward lifting force to 66 downward.

These new leaves are much stronger and more rigid than the old ones, and work very satisfactorily. The thought in the design was to build the leaves so that they would just about float, and thus eliminate the need of air. For lengths not exceeding 60 ft., the plan is very promising.

At Dam No. 6 two bear-traps were installed, each being 120 ft. long. The principal members were of steel filled in with wood. The leaves were proportioned on the basis of 100 lifting force to 80 downward. In order to assist in the raising of the gates, arrangements were made for increasing the buoyancy by air forced to the under side of the lower leaves. The air was intended to be pocketed between the ribs of the girders. This proved to be of little value, because the air could not be retained where it was needed. If both bear-traps are up and one is

FIG. 1.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

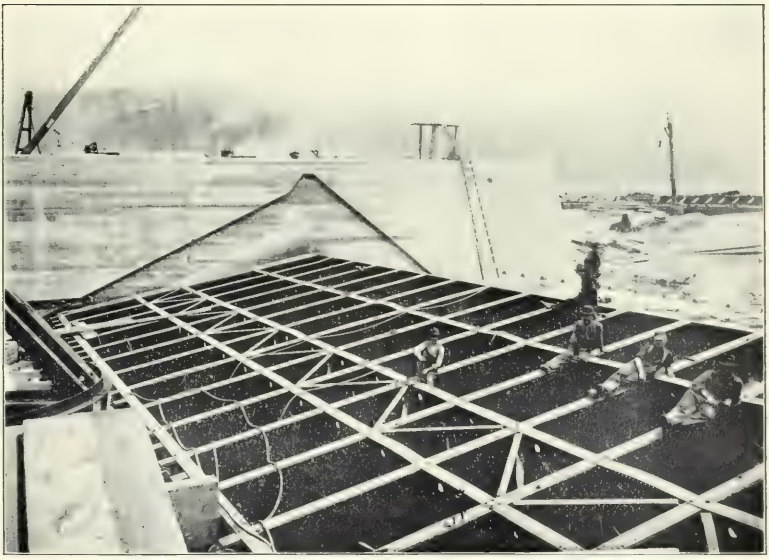


FIG. 2.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

lowered temporarily, air is not used in raising it again. It takes a head of about 2½ to 3 ft. to raise the gates, if air is used, and about 3½ ft. without it. These gates rise in from 6 to 8 min., if under a 9-ft. head, and can be lowered in from 3 to 4 min. So much of this dam is of the automatic type—about 25% of the entire dam—that it is difficult to secure the head needed to raise the gates if both are down.

Each of the lower leaves of the bear-traps in Dams Nos. 3, 4, and 5, consists of 19 girders, spaced 5 ft. apart, center to center. There are also five longitudinal braces, called girts, which divide the leaf into 72 sections. Certain girders are solid throughout, and divide the leaf into three water-tight compartments. All the girders are hinged, at the down-stream end, to steel castings anchored to the foundation. The up-stream ends are provided with rollers which support the girders of the upper leaves as they slide over the upper end of the lower leaves. Each of the steel castings, which holds the rollers, terminates in a double hook which engages in stops on the upper end of the up-stream leaf, when both leaves reach the limiting height. In order to hold the gates in the upright position during repairs, etc., locking-pin holes are provided. The skin-plating of the lower leaf consists of buckle plates and flat plates. Manholes are left in the plates, those in the upper side of the leaf, being provided with covers, are intended for use only in cleaning the leaf, making repairs, etc., while those in the lower side are intended to permit the water in the leaf to be displaced by air. The lower leaf when filled with air is calculated to have such buoyancy as will cause the entire bear-trap to rise in still water, no hydraulic lifting force being needed. The air enters the pockets of the lower leaf through special ball and socket pipe joints.

Leakage from beneath the gates, between them and the piers, is prevented as far as possible by water-seal arrangements placed so as to be pressed against the sides of the piers. At the lower end of the gates, curved plates attached to the leaves move close to similar plates attached to the hinge castings. (Plates CXXXIII and CXXXIV.)

The question of leakage prevention is of great importance, and should be carefully considered. The bear-trap gates at the Herr Island Dam in the Allegheny River could not be raised automatically until the space between the upper and lower leaves was decreased by thin boards. They can now be raised under a 4½-ft. head without the use of air.

The upper leaves of the bear-traps at Dams Nos. 3, 4, and 5, consist of a series of nineteen I-beams, spaced 5 ft. apart, center to center. These girders are separated by longitudinal braces. Each I-beam is hinged at the up-stream end to a steel casting anchored to the foundation. The leaf is sheathed with dressed white oak, carefully fitted between the I-beams. This leaf does not have to support any water pressure, and is made as described so as to decrease its weight.

The water for operating these gates is taken from the sides of Piers Nos. 1 and 3 farthest from the bear-traps. This is because there is so much fall around the heads of these piers, and it is important to use all the upward pressure that can be obtained. The conduits are built so that the water enters each bear-trap from each end. This was done in order to decrease the tendency of the leaves to warp. These traps are proportioned on the basis of 100 upward lifting force to 60 downward. They have not yet had sufficient trial to permit of the expression of opinion as to their merits. The upper end of the foundation is arranged for supporting a needle-dam, and this can be used to secure the head needed to raise the gates, if impossible to raise them otherwise, and can serve as a means of holding the pool while they are being repaired.

Chanoine Weirs.

The Chanoine weirs, which are located between the bear-traps and the abutments of all these dams except Dam No. 6, are similar to the passes. The wickets are raised, however, by a winch running on a structural-steel service-bridge, standing on the weir foundation above the wickets.

It was originally intended that the entire dam at Davis Island should be operated from a service-bridge, but before its completion it was decided to place the journal boxes for the bridge in the pass foundations, but not to erect the bridge except above the three weirs. The pass was operated from a small maneuvering boat. During a sudden freshet in 1887, the bridge of Weir No. 1 was destroyed by drift, and, in 1889, the bridge of Weir No. 2 was destroyed in the same way. On a later occasion, the bridge above Weir No. 3 was struck by a coal barge during a freshet, and partially wrecked.

The original trestles were composed of rather light members, and those built later have been made much stronger. It seems to be advisable, however, to avoid the use of service-bridges in the navigable passes of the Ohio River for the following reasons:

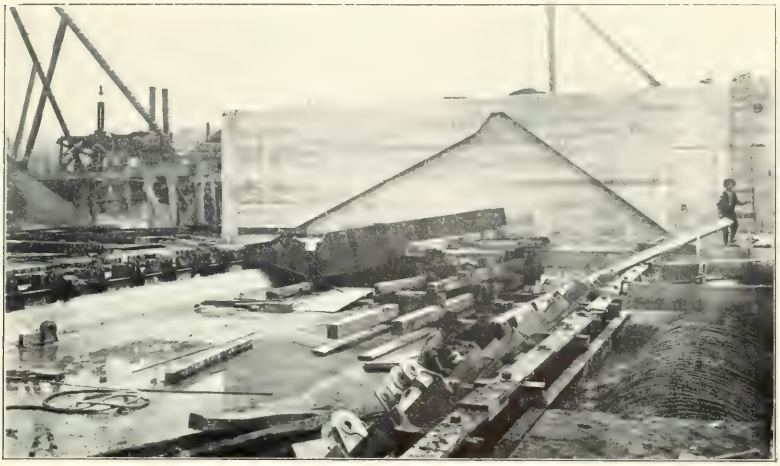


FIG. 1.—BEAR-TRAP DAM, LOCK NO. 5, OHIO RIVER.

FIG. 2.—LOCK NO. 1, ALLEGHENY RIVER.

FIG. 3.—LOCK NO. 1, OHIO RIVER.

(1).—The bridge and its foundations constitute an extra expense.

(2).—Likelihood of destruction of the bridge by drift or ice.

(3).—The bridge retards the lowering of the dam, because, after the wickets are lowered, it is still necessary to lower the bridge. The lowering of the pass can be effected quickly by a maneuvering boat.

The latest weir service-bridges for these dams have trestles 8 ft. apart, center to center, the trestles being arranged to rotate about their bottom axles, which turn in journal boxes anchored to the foundation. The down-stream journal box is open on top, the axle being prevented from coming out by an iron key. When raised, the trestles are kept in place by steel floor-panels, each hinged at one end to the top of a trestle, and held at the other end by pins attached to the adjoining trestle, the pins passing through holes in castings attached to the end of the floor-apron. The floors form a track for the winch used to operate the wickets, and serve also as a footbridge for the dam-tenders.

An effort was made to make these bridges strong enough to resist the destructive effect of drift, and they were also made strong enough to support the full upper-pool pressure, assumed to be borne by needles resting at the bottom against the oak sill in which the upper journal boxes are embedded, and at the top against horizontal beams resting against the tops of the trestles.

A rolling, hand-operated winch has always been used to maneuver the wickets at Davis Island Dam. The winch is provided with a drum parallel to the crest of the dam. Another winch having a drum parallel to the direction of the current, is used to raise and lower the bridge trestles successively.

The operating winches for Dams Nos. 2 and 5 are to be operated by compressed air, furnished from the power-house through a 4-in. pipe, embedded in the foundation of the dam and ending at the abutment.

The maneuvering boats used to operate the wickets of the passes are provided with derricks, hoisting engines, steam capstans, etc. The hull of one of the latest of these boats is 70 ft. long, by 22 ft. wide, and 3 ft. deep, and is built of wood throughout. A stiff-legged derrick was placed near the bow of the boat; this is operated by a three-drum hoisting engine.

The wickets are raised by a wire line leading from the hoisting engine around a sheave held at the end of a structural-steel beam, which projects from the bow of the boat. The wire line is fastened to

an iron hook at the end of a long pole, which is used to guide the hook until it engages in the handle of the wicket. After the prop is seated in the hurter, the wicket is assisted in seating itself against the sill by pike poles in the hands of the dam-tenders. In lowering the wickets, the upper ends are pulled slightly up stream, until the prop unseats itself, when the wicket is allowed to fall upon the foundation behind the sill.

The boat is prevented from touching the wickets above their axes of rotation by spuds which bear against the wickets below the tops of the horses. The current around the end of the wickets already raised is very swift. A manila rope, of 2 in. diameter, is fastened at one end to the river wall of the lock, and, at the other end, it is wound around the steam capstan on the stern of the boat. This rope prevents the boat from being swept around the ends of the wickets, and also serves to draw the boat back to the river wall after all the wickets have been raised.

The abutments of these dams are not subjected to the severe conditions which exist at the usual fixed dams, because the lifts are smaller; and when a freshet comes, the dams are lowered, thus restoring approximately the normal discharge area of a stream. However, it is necessary to design the abutments so that they will be free from danger of failure, due especially to eddies produced by the flow of water through the bear-traps and by the current of water flowing through the weirs. If the abutments are not founded on rock, they rest on bearing piles, and are protected, not only by a row of Wakefield sheet-piling, but also, below the dam, by stone-filled cribs placed in front of the sheet-piling. The abutments are of the U-type, with a wall extending down stream, the function of which is to prevent the washing away of the bank below the abutment. It is deemed necessary to carry this wall for a distance of from 100 to 200 ft. below the crest of the dam, depending on the character of bank material, value of property, proximity of buildings, etc.

The operation of such a system of movable dams as that described, demands a very careful watch of the discharge of the streams upon the head-waters. This is provided for by reports received daily, or more often if necessary, from observers stationed at various points. It will probably be necessary to construct a Government telephone or telegraph line along the river, as has been done along the canalized Great Kanawha. Any increase in the low-water flow beyond that necessary to

furnish water needed for evaporation and operation, adds to the difficulties of the operation of such a system.

In operating a single dam, there are many things to be contended with. At Davis Island, boats have been known to hit a pier, a bear-trap gate, and lock-gates; there have also been many collisions with wickets. The fact that a floating coal barge destroyed a portion of the service-bridge has been noted. However, if enough spare wickets, horses and props are kept on hand, those injured can be replaced by a diver without much difficulty. Some of the movable parts were broken by the stern wheels of steamboats before the dams were in operation, and it is very necessary to provide a sufficient number of spare parts. Notwithstanding all difficulties, Davis Island Dam has been operated successfully for twenty-two years, and Dam No. 6 for three years.

The 150 pass wickets of Dam No. 6 have been raised in 1 hour, 40 min., the river being at a natural stage of 6.4 ft., and the dam has frequently been raised in $2\frac{1}{2}$ hours. This dam can easily be lowered in 1 hour. (Fig. 3, Plate CXXXIV.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE SINKING OF THE PIERS FOR THE GRAND TRUNK PACIFIC BRIDGE AT FORT WILLIAM, ONTARIO, CANADA.

Discussion.*

By Messrs. C. R. Fickes, and George B. Francis and Robert A. Marshall.

Mr. Fickes.

C. R. Fickes, M. Am. Soc. C. E. (by letter).—This paper is of especial interest to the writer, who recently completed the deep foundation work for a bridge over Red River near Shreveport, La., for the St. Louis Southwestern Railway Company.

The weather conditions were not so frigid as those described by Mr. Wiley, but such conditions would have been welcomed in view of a yellow fever epidemic that occurred during the construction of the bridge.

Seven piers and one abutment were founded by the open-caisson method. Four of these, including the pivot pier, were carried down to solid rock, while the others were built on piles driven under water by a long follower.

The method of founding the pivot pier may be of interest.

This pier was in the channel of the stream which had a velocity of about 4 miles per hour at ordinary stages of the water and 8 miles at high-water stages. There is a variation of about 40 ft. between extreme high and low-water stages, and the tropical rains of that region often bring on a rise of 20 ft. or more in 12 hours.

*This discussion (of the paper by H. L. Wiley, Jun. Am. Soc. C. E., printed in *Proceedings* for August, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Fickes.

Borings previously taken at the site of this pier showed that the rock was about 90 ft. below the ordinary stage of the water and was overlaid by a stratum of stiff blue clay 20 ft. thick. Above this, up to the bed of the stream, the borings showed ultimate layers of clay, gravel and coarse sand for a thickness of 40 ft. This made about 65 ft. of soil to be penetrated by the caisson.

The coffer-dam was octagonal in shape, being 37 ft. on the short diameter. It was built of heavy timbers of yellow pine varying in size from 12 by 12 in. in the bottom courses to 8 by 12 in. at the top of the crib. These timbers were fastened together by ¾-in. drift-bolts, 4 ft. apart in each course, and the crib was braced inside by 6 by 12-in. sticks joining the centers of alternate sides of the octagon. These braces were from 4 to 6 ft. apart vertically and also served as supports for a cylindrical movable form to be described later.

The cutting edge of the caisson was formed of ⅝-in. plates, 30 in. wide, joined at corners with 4 by 4-in. angles and well riveted. The cutting edge was fastened to the bottom courses of the timbers with heavy iron straps and ¾-in. bolts.

All joints between timbers were carefully caulked with oakum before being lowered beneath the water.

Vertical sheeting of 2 by 12-in. surfaced planking was spiked on the outside of the crib to lessen the friction.

The caisson was built on a platform supported by piling driven circular around the site of the pier, and lowered to the bed of the stream by eight lowering screws 3 in. in diameter and 30 ft. long.

Inside this octagonal caisson, and concentric with it, was supported a cylindrical movable form 25 ft. in diameter, made of 2 by 6-in. by 12-ft. lagging, smooth on outside of cylinder and well braced inside. This left a space about 5 ft. wide between the form and inside of caisson, which was filled with concrete as the caisson was lowered. The form was raised as soon as the concrete hardened and was not allowed to become submerged in the water.

This wall of concrete gave the requisite strength to withstand the pressure when the crib was pumped out, and also the necessary weight to overcome friction in sinking.

This concrete shell was carried up 65 ft. from the cutting edge, so that the top of it would correspond with the bed of the stream, when the cutting edge had reached the rock.

When the caisson had been lowered to the bed of the stream, the material inside was removed by two 1-yd. orange-peel dredges operated by derricks and hoisting engines mounted on a large floating barge anchored beside the crib. Other barges carrying the concrete mixer and material were anchored on the opposite side of the crib, and the operations of building up the concrete and timber wall and of excavating and sinking the crib went on alternately.

Mr. Fiekes. It was found that the weight of the wall was sufficient to carry the crib down to the required depth and no additional loading was necessary.

Occasionally, when the excavation had been carried down 3 or 4 ft. below the cutting edge and the crib failed to follow, a small charge of about half a stick of dynamite, dropped into the hole and exploded, caused the crib to settle at once. Several logs were encountered by the cutting edge. These were broken up by dynamite placed by divers and exploded by an electric battery.

No trouble was had at any time from material running in from the outside under the cutting edge.

Only one mishap occurred during the construction of this pier, and that happened when the cutting edge lacked only a few feet of reaching the rock. The top of the concrete wall was then about 3 ft. above the bed of the stream, and above this was about 25 ft. of timber crib extending a few feet above the water. A sudden rise in the river brought down several acres of driftwood, carrying away four clusters of protection piles which had been driven up stream from the pier, and finally breaking off and carrying away that part of the crib above the concrete wall. All barges and floating equipment were torn loose from their anchorages and only recovered with much difficulty several miles down stream.

The problem now was to replace the lost part of the crib and attach it to that portion left intact under about 20 ft. of water. This was accomplished as follows:

A new crib of the same shape as the old, but 2 ft. less in diameter, was built and floated into position over the submerged portion, and, by means of weights, sunk so that its lower edge rested on the concrete wall just inside the old timber part of the crib. On the lower edge of this new crib was placed a pad of felt, 2 in. thick, for the purpose of partially caulking the joint. The new crib was securely fastened to the old one by $\frac{3}{4}$-in. straps and spikes driven by divers, who also completed the caulking of the joint.

When this was accomplished, the excavation of material from the inside of the crib was continued by the dredges, and the cutting edge was soon lowered to the rock.

Divers were then sent down inside the caisson, and the surface of the rock, which was found to be fairly level, was cleaned off as thoroughly as possible by water jets. A hydraulic sand pump was also used for removing the material stirred up by the jets.

A layer of 1:2:4 concrete, 4 ft. thick, was then deposited over the surface of the rock by bottom-dump buckets carefully lowered through the water. This concrete was placed without any interruption so that there would be no joint. Forty-eight hours was given for this concrete to harden, after which two 8-in. pulsometer pumps were lowered into

the hole, and unwatered the caisson in a few hours. The bottom was found to be sealed effectually by the layer of concrete already described, and the shell of concrete was practically waterproof. Some little leakage occurred at the joint between the old and new parts of the crib, but this was soon overcome by caulking. Mr. Fickes.

After this, the concreting proceeded rapidly in the dry, and the well was soon filled up to the point where the neat work was to begin. The forms were then erected and the pier carried to completion without mishap.

The average daily distance of sinking the caisson was about 4 ft., and the greatest distance sunk in ten consecutive hours was 8 ft. This is exclusive of time lost by reason of high water and replacing the crib as described.

The completed pier is 26 ft. in diameter above the water and contains approximately 3 000 cu. yd. of concrete. It is the support for a 300-ft. single-track railway swing span.

The cost of the completed foundation was much less than the average cost of pneumatic work and has several advantages not possessed by the latter, among which is the fact that the continuity of the concrete is not broken by a deck of timber as is the case with pneumatic foundations.

No trouble was experienced in keeping the caisson in position during the sinking. By careful work with the dredges, the crib was kept practically level at all times. When finally landed, it was less than 2 in. off center in either direction.

The other caissons sunk by this method were rectangular, 40 by 22 ft., and had concrete walls inside the timber cribs, 4 ft. thick, with a cross-wall of concrete dividing the inside into two wells 12 ft. square, through which the earth was excavated as from the single well of the octagonal caisson.

No trouble was experienced with any of these caissons, as no sudden rise occurred in the river during their construction and sinking.

In the case of one pier erected on shore, the caisson was carried 85 ft. through the soil. Weight in addition to that of the concrete wall was necessary in this case and was secured by loading the crib with large boxes filled with earth removed by the dredges from the inside of the crib.

The completed bridge consists of four through spans of 200 ft. each, one of 150 ft., and a swing span of 300 ft. There are about 100 ft. of creosoted pile and steel trestle approaches at each end of the bridge.

J. W. Schaub, M. Am. Soc. C. E., was Consulting Engineer for the work, and prepared the plans. M. L. Lynch, M. Am. Soc. C. E., is Chief Engineer of the St. Louis Southwestern Railway. The contract for foundations and steel erection was held by the Missouri Valley

Mr. Fickes. Bridge and Iron Company, the steel being fabricated by the Phœnix Company. The writer had direct charge of the work as Resident Engineer from start to completion.

Messrs. Francis and Marshall. GEORGE B. FRANCIS and ROBERT A. MARSHALL, MEMBERS, AM. SOC. C. E. (by letter).—The description of this foundation work possesses more than usual interest from the fact that it is a successful execution at or near the extreme depth below water, at which open coffer-dam work is practicable. From general observation of such work, no case comes to mind where such depths have been reached by open coffer-dam, either in open water or through strata beneath such a depth of water. The common limit for such work is 50 ft. or less; beyond that depth pneumatic caisson methods are usually adopted. The question might be asked why the open method was selected in preference to the pneumatic, or why a pile foundation was not used.

The description has also proved very interesting from the fact that recently the writers were called on to design a method of sinking or placing piers at quite similar depths, in the Ohio River. Two methods were outlined, which, although not used, may nevertheless prove interesting.

In the study on these designs the writers found two recent instances —one, the Pyrmont Bridge, at Sydney, Australia, the other at Bristol, England—of the founding of similar piers in deep water, which have been fully described.*

A comparison of the case described in the paper with the Ohio River designs, the Sydney Bridge and the Bristol Bridge, taking typical piers, would be as given in Table 1:

TABLE 1.

Name.	Size of pier, in feet.	Depth of base below highest water, in feet.	Depth of base below bed of stream, in feet.	Base rests on:	Material of pier.	Method.
Fort William Bridge	31 diam.	60	±40	Rock.	Concrete in steel shell.	Steel shell sunk and pumped out.
Ohio River Bridge..	35 by 80	48	±36	Rock.	Concrete and stone.	Steel sheet-piling, and pumped out.
Sydney Bridge.....	42 diam.	60	±36	Rock.	Concrete in steel shell.	Wrought-iron shell sunk and pumped out.
Bristol Bridge.....	43 by 37	45	±17	Marl.	Concrete and stone.	Timber sheath-pile coffer-dam.

Proposed Methods of Placing the Piers for the Ohio River Bridge. —The proposed bridge was designed to have three piers of an average

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXX, Dec., 1907.

Messrs. Francis and Marshall.

size of 35 by 80 ft., founded on rock about 36 ft. below the river bottom, with a depth of water ranging from about 1 ft. at extreme low water to about 41 ft. at record flood level.

On account of the magnitude of the work, the time required for construction would necessarily extend through the seasons productive of the maximum variation of depth of water.

The flood records in the Pittsburg office of the Weather Bureau were examined and plotted, and it was found that while the record flood of 1907 rose 41 ft. above low-water level, the rise and subsidence of water was rapid and the majority of the peaks were not more than 10 ft. above low-water level.

A coffer-dam high enough to keep out the record flood of 1907 would be impracticable. It was found that a coffer-dam high enough to keep out an 11-ft. rise of water would permit work on the foundations about 330 days per year, and give practically no interruptions from high water between April 15th and December 1st.

OHIO RIVER BRIDGE
DESIGN FOR CRIB COFFER-DAM

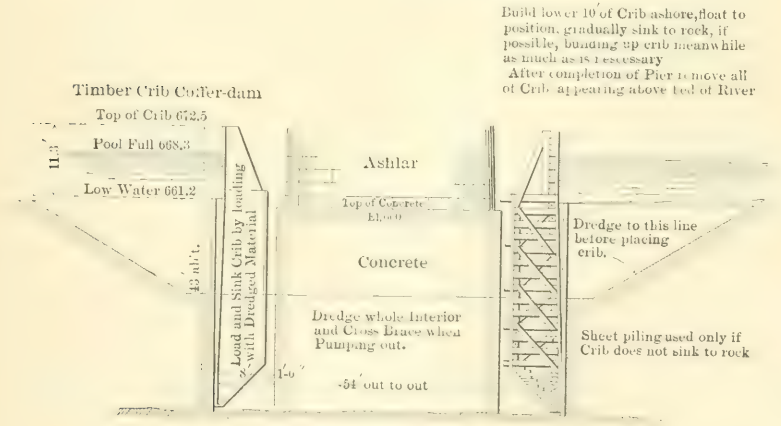
Build lower 10′ of Crib ashore, float to position, gradually sink to rock, if possible, building up crib meanwhile as much as is necessary
After completion of Pier remove all of Crib appearing above bed of River
Timber Crib Coffer-dam
Top of Crib 672.5
Pool Full 668.3
Low Water 661.2
Ashlar
Top of Concrete El. 640
Concrete
Dredge to this line before placing crib.
Load and Sink Crib by loading with Dredged Material
Dredge whole Interior and Cross Brace when Pumping out.
Sheet piling used only if Crib does not sink to rock
1′-0″
54′ out to out

FIG. 2.

The two principal requirements of construction were that the rock should be uncovered for inspection and then leveled or stepped; and that the concrete foundation should be a solid monolithic mass under the entire pier.

The first requirement made it impossible to deposit concrete under water, as was done in the Fort William Bridge; the second required the protective works to be entirely outside the lines of the pier.

The first plan consisted of the use of a timber crib, the bottom section of which would be built ashore, launched, and towed to the site,

Messrs. Francis and Marshall.

which, in the meantime, was to be partially dredged. The crib, after being securely moored, was to be sunk by loading with material dredged from the river bottom. It was designed with a **V**-shaped bottom, and the sinking was to be continued by dredging the interior, building up the sides, and continuing the loading until it rested on rock. In construction, the crib was to be made of squared timbers, laid cob-house and thoroughly braced and fastened with dock spikes. The outside and inside were to be covered with tongued and grooved sheathing well driven up, which would give better protection than two lines of sheet-piling, without any of the irregularities from obstacles frequently encountered in driving. The interior was then to be pumped out, and cross-bracing between sides added as the water was lowered. Should unevenness in the rock surface cause a blow or serious leak, it was proposed to stop it by driving outside sheet-piling at the point where needed.

OHIO RIVER BRIDGE
ALTERNATE DESIGN FOR COFFER-DAM

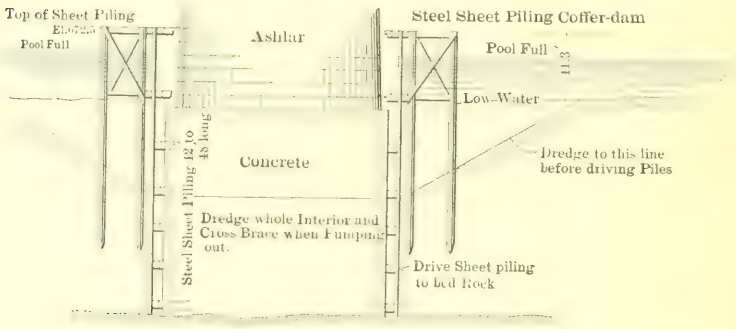
Top of Sheet Piling
Pool Full
Ashlar
Steel Sheet Piling Coffer-dam
Pool Full
Low-Water
Concrete
Dredge to this line before driving Piles
Steel Sheet Piling 42 to 48 long
Dredge whole Interior and Cross Brace when Pumping out.
Drive Sheet piling to bed Rock

FIG. 3.

After cleaning and leveling the bottom, concreting was to commence, and the cross-bracing was to be removed as fast as the concrete was placed.

As an alternative to the method outlined above, a coffer-dam made of interlocking steel sheet-piling was considered, as it is possible to drive such piling to rock at all points and thus have a tighter job around the bottom, than is the case with a crib which must necessarily stop at the highest point of rock.

The material at the site consisted generally of compact sand and gravel, and, by partial dredging, the cost of driving could be materially reduced.

The progress, as outlined, consisted of dredging to a depth of 15 to 20 ft., driving and bracing the necessary guide piles which would also

Messrs. Francis and Marshall.

support the working platform in a manner similar to that used in the Fort William Bridge piers; then driving to rock a single line of interlocking steel sheet-piling weighing 45 lb. per sq. ft. The coffer-dam would then be pumped out, the sheathing being securely braced as the water was lowered. The remaining material inside the coffer-dam would be dredged to rock and concreting commenced, the interior bracing being removed in advance of the concrete.

When the pier was completed to the top of the coffer-dam, the sheet-piling would be pulled and used again on the next pier.

After careful consideration of both methods, it was decided to recommend the use of the steel sheet-piling, the estimates prepared showing it to possess a larger element of economy than the crib construction.

The manufacture of steel sheet-piling is of comparatively recent development and it possesses advantages in foundation construction which, heretofore, were not available.

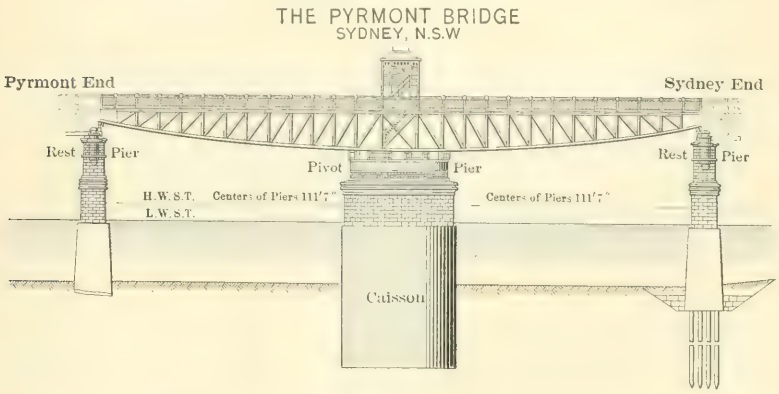

FIG. 4.

Method of Placing a Typical Pier for the Sydney Bridge.—The caisson, which formed a part of the pier as well as a coffer-dam, was a double wrought-iron shell with an external diameter of 42 ft., an internal diameter of 32 ft. and a total height of about 53 ft., the shells being concentric and brought together at the bottom to form a cutting edge at the line of outer diameter. This caisson was sunk to the river bed, then loaded with concrete and, while the interior was being dredged out, the caisson was sunk to bed-rock.

The water was then pumped out, but a blow-out occurred which necessitated the excavation for the leveling off of the irregular rock surface by the aid of divers, with the caisson full of water. Some of the concrete in the base, around the outer margin of the circle, was placed by divers. After this no trouble was experienced in pumping out the caisson, and the remainder of the work of leveling and of placing

Messrs. Francis and Marshall.

concrete was done in the open air. From a point just below low water, the concrete pier was carried up with a facing of cut stone. The time occupied in sinking the caisson was nine months.

The thickness of the wrought-iron shell used in the caisson varied from $\frac{1}{2}$ to $\frac{5}{16}$ in., and the concentric rings were connected with angle-bar bracing.

This bridge was opened for traffic in June, 1902.

Method of Placing a Typical Pier for the Bristol Bridge.—The method adopted was to drive a timber coffer-dam of a single line of 12-in. material, between guides, and to caulk this with oakum, making a dam to hold back 28 ft. of water at high water and about 2 ft. at low water.

PYRMONT BRIDGE, SYDNEY, AUSTRALIA

BASE OF CENTER PIER

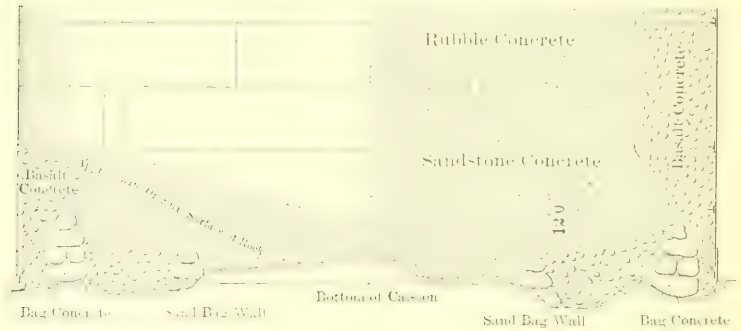

FIG. 5.

Excavation was made in the open without trouble to a point about 6 ft. below low water, when several blow-outs occurred at times of high tide, after which excavation was carried on only at low tide. Concrete foundations were placed in air, at low-tide periods, to low-water elevation, and, above that elevation, the concrete core was faced with cut stone and laid up in the open air at all stages of the tide. A double line of timber sheathing for the coffer-dam was not practicable on account of the waterway required for navigation.

RIVER AVON BRIDGE, BRISTOL, ENGLAND

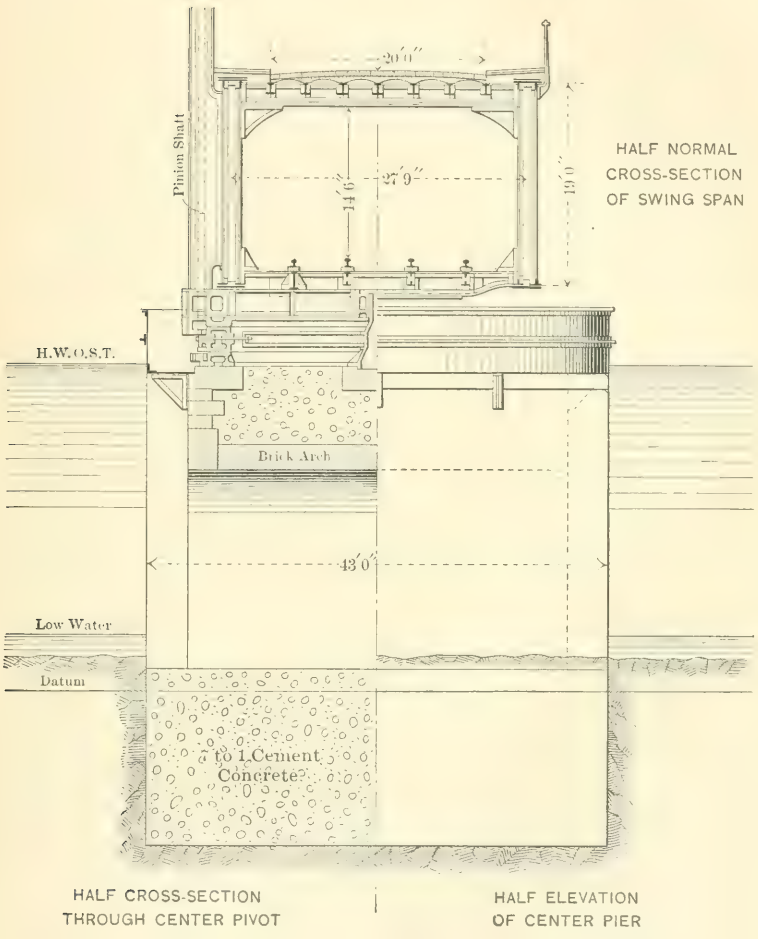

FIG. 6.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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CATENARY TROLLEY CONSTRUCTION.

Discussion.*

By Joseph Mayer, M. Am. Soc. C. E.

Mr. Mayer. Joseph Mayer, M. Am. Soc. C. E. (by letter).—This paper ably presents the results of the latest, practical, American experience with catenary trolley construction. The main features of the line described are: A 0000 phono-electric contact wire of one and two-thirds times the elastic limit of the best hard-drawn copper wire, with 2 000 lb. tension, at 100° fahr., 120-ft. spans of the carrying ropes on tangents, and shorter spans on curves, and solid round hangers 10 ft. apart.

By calculating the bending strains and tensions in the contact wire, the writer has reached, and published, the conclusion that a really safe catenary suspension for high speeds is impracticable with a copper contact wire without introducing adjustments of the length of the wire to balance the effects of changes of temperature. The large bending strains and tensions in the contact wire cause the practical failure, with high speeds, of the older catenary suspensions. In the United States, the remedies are the use of a stronger wire, and, in some cases, of a supplementary wire, and of short spans of the carrying ropes; in Europe, a supplementary wire and complicated adjustments for changes of temperature are used. The supplementary wire of the German Siemens-Schuckert suspension carries the contact wire in loose loops, and secures, without excessive tension in the contact wire, on the regular line, an approximately straight motion of the current collector. The jumping of the latter is thereby avoided.

* This discussion (of the paper by Oliver S. Lyford, M. Am. Soc. C. E., printed in *Proceedings* for August, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Mayer.

For obtaining continuous contact at high speed, the usual single-wire catenary suspensions require a light current collector and a large minimum tension in the contact wire. With a drop of 140° in the temperature of the wire, this tension, with a copper wire, is increased about 21 000 lb. per sq. in. The deflections of the wire produced by the wind pressure, the pressure of the current collector, ice loads, and changes of temperature, result in another increase of the tension, and, at the steady strains especially, in considerable bending strains. The maximum strain in the wire, therefore, exceeds the elastic limit of the best copper wire. Since the bending strain in the wire is inversely proportional to its least radius of curvature, it can be reduced by making the wire bend in curves of long radius; since the tension, with a given span, is inversely proportional to the sag, it can be reduced by increasing the sag. This has suggested to the writer a new method for obtaining a safely suspended wire, namely, by suspending it, with a large sag, from such hangers as will make it bend in curves of long least radius. The large sag gives a simple automatic adjustment for changes of temperature, because the wire can shrink on account of cold, with reduction of the sag, and without excessive increase of its tension. By such a suspension of the contact wire from a tapering steel bar, 6 ft. 7 in. long,* and by omitting the carrying ropes and the intermediate suspenders, the tensions and bending strains in the wire, with 120-ft. spans, can be reduced to less than one-half the amounts resulting from catenary suspension with the same span between the main supports. The current collector, instead of oscillating rapidly, will then move up and down slowly in long shallow waves. The reduction of the contact pressure, due to centrifugal force at high speed, is then trifling and its increase moderate, and the latter occurs under the hangers where the wire is reinforced by them. A small static contact pressure, with reduced wear of the wire and the current collector, becomes thereby practicable. This suspension is equally suitable for trolley wheels and sliding bows, while catenary suspensions are not well adapted to the former. With 0 copper wires, 220-ft. spans give a safe suspension, requiring current collectors 7 ft. long; with phono-electric wires of the kind described by the author, 300-ft. spans with an average sag of 3 ft., can be used safely on tangents. The loads and wind pressures per linear foot carried to the brackets are about one-third of those with single catenary suspensions. Poles and brackets of 120-ft. catenary spans are adequate for the 300-ft spans of the new suspension. Curves call for the same hangers and shorter spans, but, on account of the reduced tension and wind pressure and the consequently diminished side pull, much longer spans than those used by the author become practicable.

* One form of such a bar is shown in *Transactions*, Am. Soc. C. E., Vol. LXI, p. 31.

Mr. Mayer. Mr. Lyford states that for high-speed operation the maximum flexibility should be maintained at all points, and that heavy weights and hard spots should be kept out of the system wherever possible. What is here called for is impossible and unnecessary. An accurate definition of what is really needed is of much importance for obtaining a good track for a high-speed current collector.

The line should be built so that the current collector will move in curves of long least radius. With sliding-bar collectors, the vertical curvature of the motion of the collector is alone important; with trolley wheels its horizontal curvature must be equally considered. The collector exerts a normal pressure on the contact wire; this should be as uniform as possible. The centrifugal force arising from the curved motion of the collector and of the rigidly connected parts of its supporting frame produces an equal variation of the contact pressure. Where the motion of the collector is convex upward, the contact pressure is increased; where it is convex downward, it is decreased. In the former case excessive bending strains in the wire and large local wear of it may result; in the latter, there may be interruption of contact and consequent sparking. The excessive curvature of the motion of the collector may arise from too much flexibility of the contact wire; it will often arise from a sudden change in flexibility. A weight moving in a curve produces a centrifugal force, $C = \frac{W}{32.2}\frac{v^2}{r}$, where W is the weight, v the velocity, in feet per second, and r the radius of curvature of the motion, in feet. The current collector and parts of its rigidly-connected supporting frame do not move in the same manner, the radius, r, being variable. The weight, W, which produces, when moving like the contact point, the same centrifugal force as the collector and its frame, is called the equivalent weight. This equivalent weight and the radius, r, of the curvature of the motion of the contact point determine the variation of the contact pressure due to centrifugal force. The radius, r, depends largely on the design of the overhead line, but is also influenced by the static contact pressure and the size of the product, Wv^2. An overhead line is only safe and adequately smooth up to a certain maximum value of this product, therefore, this must be known in order to judge as to the adequacy of the line. The value of r must be calculated from the nature of the overhead line and the given value of the static contact pressure and of the product, Wv^2. To neglect the calculation is to incur risk of failure, unless the safety of the design and the smooth running of the collector can be inferred from previous experience, similar or less favorable in all essential points. Catenary suspensions make no satisfactory provision for harmlessly changing the direction of the overhead line and of the motion of the current collector. Such change is unavoidable at the main supports

and, to a larger extent, at low overhead crossings and tunnels where the line must be lowered and raised. Sparking or excessive contact pressure and large bending strains in the wire generally result from high speed at these points. They are weak spots, and are rarely mentioned in descriptions. On the regular line the sharpest curvature of the wire and of the motion of the collector, in the author's design, occurs near the hangers and steady strains. In this design, the wire has 10-ft. spans, with the ends nearly fixed in direction. If the wire were practically hinged at the hangers and the steady strains, these bending strains and curvatures, and the consequent risk of jumping of the collector at high speed, would be greatly reduced. Hangers made of flat bars, $\frac{1}{8}$ in. thick, with proper end connections, which are used on some lines, provide practically hinged clamps for the wire; they are preferable, therefore, for high speeds. With a light current collector and moderate speeds, this defect of the hangers used by Mr. Lyford can be partly neutralized by using a larger minimum tension in the contact wire than would otherwise be required. This will give smooth running of the collector, but will result in larger tensions and bending strains in the wire and larger side pull on curves. The current collector raises an advancing wave in the contact wire. The wire in front of the collector turns up and rises; in the rear, it turns back and falls. If the clamps holding the wire were light and if each was connected to its hanger by a horizontal bolt, normal to the wire, passing through a slotted hole, permitting the clamps to turn and to rise when the collector passes, the hangers would not prevent a practically straight motion of the collector, and chattering would be avoided with the highest speeds and with moderate tension in the contact wire. Mr. Mayer.

This remedy is impracticable at changes of vertical slope. The wire carries a part of the wind pressure and some vertical shear to the steady strains and is subjected to consequent bending strains. These become large with long spans and compel the use of short spans. If the steady strains are replaced by flexible, tapering bars holding the wire under them, these bending strains and sharp vertical and horizontal curvatures of the contact wire are reduced to small amounts, and long spans become safe. A 0000 phono-electric wire suspended from a steel-wire rope with a diameter of $\frac{3}{4}$ in., the former with 2-ft., the latter with 5 ft. 6 in., maximum vertical sag, gives, with 300-ft. spans, with proper hangers and with flexible bars for steady strains, a safe catenary suspension for the highest speeds, suitable for sliding bows.

The flexible bars also furnish a satisfactory means of overcoming all difficulties arising from irregularities of the line at tunnels and overhead crossings. Such long spans are heavy and exposed to large wind pressure; they would have to be carried by substantial steel or reinforced concrete posts with concrete foundations.

Mr. Mayer. If a wire is hung, without carrying ropes, in straight rigid clamps, it is bent sharp at their ends. To reduce these bends and the consequent blows at high speed from the current collector, short spans and small sags of the wire become necessary. If, instead of a rigid clamp, a long flexible bar is used, the wire being attached all along the bar, and the bar being of such shape that it will bend vertically and horizontally approximately in circles, then the radius of least curvature of the motion of the collector becomes very long, and long spans with large sags can be used. Any desired change in the direction of the wire can be made by using such bars without large bending strains in the wire or excessive contact pressure. All irregularities in the line, which, in most suspensions, are the most serious sources of trouble, can be provided for satisfactorily. It can be easily shown that the cost of construction and maintenance of safely suspending the contact wire by such flexible hangers is only about half as much as with safe catenary suspensions.

For a certain maximum value of the product, Wv^2, not stated by the author, the design described by him gives, with adequate poles and brackets, a safe and suitable suspension for curves and tangents. No information is given for other irregularities of the line. Wherever the value of the product, Wv^2, is small enough to give smooth running of the current collector, Mr. Lyford's design is decidedly preferable to the Siemens-Schuckert suspension, which is more complicated and more expensive to construct and maintain. The safe value of the product, Wv^2, and, therefore, the limits of usefulness of the design have evidently not been determined by the author.

In the writer's opinion, the introduction of carrying ropes and numerous hangers to suspend the wire at short intervals in an approximately straight line is a mistake. It complicates the design, calls for short main spans and a very strong large wire, and it produces a rapidly oscillating and sharply curved motion of the current collector with consequent large variations of the contact pressure. The suspension, therefore, is unnecessarily expensive to construct and maintain. These complications were introduced to remove the sharp bending of the wire at the ends of the hangers. This can be more easily and efficiently accomplished by spreading the bending uniformly over a sufficient length of the wire. The amount of curvature at the ends of the hangers with a given span and sag is extremely variable. It depends on the wind pressure, the pressure of the current collector, the ice load, and the temperature. The long transition curve at the hangers, therefore, must also be variable, and it can only be simply produced by connecting the wire to a long elastic bar. To make the bar curve uniformly, it must have cross-sections decreasing in a suitable manner from the center toward the ends. Calculation shows that a nickel-steel bar consisting of two truncated cones and a short central

cylinder, is both sufficiently strong and flexible to meet all require- Mr. Mayer.
ments of high-speed operation on curves and tangents with an ample factor of safety, and to give, at the time of maximum tension, small bending strains in the wire. With the long spans thereby made possible, the current collector does not lift the wire above its supports; it therefore diminishes its main curvature, except under the hangers. With the frequent hangers of the catenary suspensions, the wire is lifted above the supports, and the collector moves in short waves. To judge the radii of curvature of the long and the short waves, it is well to consider the heights of waves of different lengths having the same radius of curvature. These heights are as the squares of the lengths. Regular waves, 300 ft. long and 48 in. high, have the same curvature as waves 10 ft. long and 0.0533 in. high. The motion of the collector with the suspensions here considered is in both cases a very irregular wave motion; the above equivalent heights give, therefore, only a roughly approximate idea of equivalent deflections of the wire for the two kinds of suspension. An impracticable tension of the contact wire would be needed to limit the height of the small waves to the amount above given. The comparison clearly shows that a large sag of the wire with long spans is not necessarily an obstacle to the smooth running of the collector at high speed, and to small variation of the contact pressure. It also shows that the vertical alignment of the numerous supports of the contact wire must be very accurate to avoid large increases in its curvature and in the variation of contact pressure at high speed. With the long spans an error of 6 in. in the position of a hanger produces but small additional strains in either the wire or the hanger. The erection of the long spans, therefore, requires little accuracy; 300-ft. spans, with phono-electric wires, require a maximum deflection of 4 ft. and a nearly corresponding variation of the height of the current collector. If this is objectionable, 200-ft. spans with 2-ft. sag can be used. With these shorter spans, a sliding bow 4 ft. long is adequate. Heavy weights in the line are not harmful if provision is made that they will not cause sharp curvature of the motion of the current collector. A modification of the flexible bar here proposed can always be adapted to achieve this end. A line safe for high speed through its whole length becomes thereby practicable.

The advantages of long spans between main supports are especially conspicuous where wooden poles stuck into the ground are not considered a sufficiently substantial and durable carrying structure. When concrete foundations and creosoted poles are used, the first cost of the supports is greatly increased, and the saving of half of them becomes immediately important; and still more so where steel or reinforced concrete poles are used. In such cases still longer spans are desirable; 352-ft. spans give an amply safe structure with a 00 galvanized-steel

Mr. Mayer. contact wire. The risk of damage by derailments is greatly reduced by the use of fewer supports. A flexible bar suspension requires only one kind of hangers, brackets, and poles, and the number of spare parts needed for quick repairs, therefore, is small. The small number of supports allows the choice of substantial and durable ones without excessive cost of the line.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES ISAAC HAYCROFT, Assoc. M. Am. Soc. C. E.*

DIED APRIL 11TH, 1908.

James Isaac Haycroft was born in Cork, Ireland, on May 25th, 1853. In 1869 he commenced the study of engineering at Queen's College, Cork. After a distinguished career as a student, he obtained the degree of Bachelor in Engineering in 1872; and, in 1882, the degree of Master of Engineering was conferred upon him.

On the completion of his academic training, Mr. Haycroft entered the office of B. B. Stoney, M. Inst. C. E., of Dublin, Ireland, under whom he had unusual opportunities for acquiring experience in the design and construction of harbour and dock works in connection with the Port of Dublin; he afterward occupied positions as Assistant to the County Surveyor at Antrim and at Clonmel, and was Local Surveyor at Carrick-on-Suir. For some time Mr. Haycroft was associated with the firm of Messrs. Wells, Owen and Elwes, Civil Engineers, of London. In 1886, he left England for Australia. He spent about a year in Sydney, qualifying as a licensed Surveyor of Crown Lands and Real Property, and, in 1887, he obtained the two certificates.

For about eleven years Mr. Haycroft occupied the position of Borough Engineer of Woollahra, Sydney, and, at the same time, he carried on the practice of a Consulting Engineer. He resigned the position of Borough Engineer to accept one as Engineer in the Tramway Construction Branch of the Public Works Department, and remained in that Department until within a few days of his death.

Mr. Haycroft was a Member of the Institution of Civil Engineers of Ireland, an Associate Member of the Canadian Society of Civil Engineers, and a Member of the Society of Municipal and County Engineers, England. He was also a Member of the Royal Society of New South Wales and of several local societies, to which he contributed papers on technical and professional subjects, including bridge construction, roads, drainage, etc.

Mr. Haycroft had a wide range of scientific and technical knowledge, was a close reader, and was devoted to his profession. His death came as a great surprise and grief to his numerous friends. He married Julie S. A. Bowen, of Nottingham, England, who survives him.

Mr. Haycroft was elected an Associate Member of the American Society of Civil Engineers on January 3d, 1894.

* Memoir prepared by Professor W. H. Warren, University of Sydney, Sydney, Australia.

623.6 William P. Morse T

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

November, 1908.

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THE FLOODS OF THE MISSISSIPPI DELTA: THEIR CAUSES, AND SUGGESTIONS AS TO THEIR CONTROL.*

By William D. Pickett, M. Am. Soc. C. E.†

One of the great engineering problems of the age, if not the greatest, is to obtain such control of the waters of the Mississippi River as to prevent those periodical overflows which, in the past, have devastated and made desolate the entire area of its delta. This delta is probably the most extensive in the world, and has been formed, during past ages, by the discharge of the waters of the greatest river in the world, certainly the longest, in miles, and with a volume of discharge not exceeded by any, unless it be the Amazon, in South America.

From a geological standpoint, the most plausible theory in regard to the formation of this delta is that at one time the Gulf of Mexico extended as far north as the confluence of the Mississippi and Ohio Rivers; that, in past ages, by the discharge of the silt brought down

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

† As a matter of interest the fact is here mentioned that Mr. Pickett has been a Member of this Society since July 6th, 1853, having joined it during the first year of its existence. *Secretary.*

in the annual floods, the land has been gradually formed and extended out into this inland sea, until, about the beginning of the nineteenth century, it had assumed the form and area existing to-day.

When it is considered that the level of the low-water stage at Cairo is about 250 ft. higher than the mean tide level of the Gulf, the centuries of time required to develop this delta and the depth of its soil of almost illimitable fertility may be realized.

Following a natural law, the land on the banks of this river is higher than that farther inland. The same conditions obtain on Deer Creek and Sunflower River, in the Yazoo Basin, as on the Bayou Plaquemine and other large bayous draining the lower portion of this delta. A cross-section of the Yazoo Delta, near the latitude of Greenville, Miss., made by the Engineer of the Yazoo Levee System previous to 1861, developed the fact that the level of the bank of the main river was about 13 ft. higher than comparative levels 20 or 25 miles inland near Deer Creek and Sunflower River; such was the writer's information at the time.

As a natural sequence to these conditions, the "high-lying lands" on the main stream and such subsidiary streams as those noted above, were first brought under cultivation, and, being of unexcelled fertility, produced the heaviest yield of cotton, corn, and other crops suitable to the climate.

The area of these "high-lying," and, of necessity, more valuable lands, is a small percentage of the area of this delta. A much larger area consists of "low-lying lands," designated by the U. S. Geological Survey as "Sharkey clay," which are more or less subject to annual overflows, and are covered with a heavy growth of timber, such as white oak, red oak, water oak, cypress, and gum, all more or less valuable for commercial purposes.

The cost of clearing off this heavy growth of timber and preparing the ground, by ditching, etc., for cultivation, together with the almost certainty of periodical overflows, has heretofore and will hereafter prevent these otherwise valuable lands from being brought under cultivation and rendered productive until the levees on the main river have been perfected so as to render these overflows improbable, if not impossible.

These low-lying "overflow lands" are represented, by officers of the Geological Survey and by intelligent planters, to be of as great fertility

and as productive of corn and cotton as the "high-lying" lands on the streams, where the ground has been properly prepared. With land of this character, protected from overflow, it is believed that the value of its timber for commercial purposes will go far toward meeting the cost of preparing the ground for crops, if it does not equal or surpass it.

These facts being substantially true, it is proposed to investigate and ascertain approximately the value, to the nation, of these "overflow lands" on the supposition that they could be freed from that incubus of "overflow" so much dreaded by the Mississippi River planter.

The most reliable information as to the extent of this entire delta is obtained from the Agricultural Department, through Mr. J. A. Bonsteel, of the Bureau of Soils, who from the most reliable records accessible estimates this area at 30 526 sq. miles. From the surveys of that Bureau in the Yazoo Delta, it is ascertained that 65% of the total area is "overflow" land, leaving 35% for land under cultivation. Adopting this basis for the entire delta, it indicates the total area to be, in round numbers, 30 000 sq. miles, and the area of "overflow" lands to be 20 000 sq. miles of land of the highest fertility, yet it is of no value, and never will be, for crops, for causes before described.

From the writer's knowledge of the conditions in this delta, it is believed that the land under cultivation is nearer 25% of the total area than 35 per cent. The 35%, however, has been adopted in the following estimate furnished by Mr. Bonsteel:

"Area of Mississippi Delta, in Square Miles, by States:

"State.	"Total area	"Area of overflow land.
"Kentucky	150	100
"Tennessee	398	260
"Missouri	3 111	2 022
"Arkansas	5 406	3 513
"Mississippi	6 966	4 425
"Louisiana	14 495	9 425
	"30 526	19 745."

In making an estimate of the yield of this now non-productive land, the area of Kentucky, Missouri, and 100 sq. miles of Tennessee is assigned to corn; 160 sq. miles of Tennessee and all of Arkansas and Mississippi are assigned to cotton, and all of Louisana is assigned to

sugar and rice. As the necessary statistics for the yield of Louisiana lands have not been attainable, a yield of $35 per acre has been estimated, the same as the yield of cotton lands.

To compensate for lands not available for crops in the "overflow" area, such as that occupied by rivers, bayous, and creeks, 10% has been deducted. The results, in round numbers, are as follows:

Corn land.....	2 000 sq. miles	= 1 280 000	acres.
Cotton land.....	7 300 " "	= 4 672 000	"
Sugar and rice lands...	8 500 " "	= 5 440 000	"

In placing a value on crops from these lands, the yield of corn has been taken at 35 bushels per acre, and the value at 50 cents per bushel. The yield of cotton has been taken at $\frac{7}{8}$ bale per acre and the value at $40 per bale. The yield of sugar lands has been taken at $35 per acre, the same as cotton lands.

Turning these statistics into dollars and cents, we have an estimate —conservative, it is thought—of the value of crops that can be realized from land that never has had and never will have much intrinsic value for agriculture until freed from that overhanging incubus—the periodical overflows from that mighty river. The little benefit that has been realized in the past, from this overflow land as a range for stock, has been much more than counterbalanced by the destruction of stock in such overflows.

1 280 000 acres of corn land at 35 bushels per acre = 44 800 000 bushels at 50 cents.....	$22 400 000
4 672 000 acres of cotton land at $\frac{7}{8}$ bale per acre = 4 088 000 bales at $40 per bale.....	163 520 000
5 440 000 acres of sugar land at $35 per acre.....	190 400 000
Total value of crops.....	$376 320 000

Bear in mind that this estimate is based on the yield of fresh land, which should be much greater than that from the old plantations of this delta which have been under continuous cultivation for 50 or 60 years, or more. Bear in mind, also, that it is based on what the ground will bring forth with average cultivation, not on the profit to the planter. Bear in mind, also, that this estimate is exclusive of the 35% (9 000 sq. miles) of this delta which is supposed to be under cultivation and already producing crops. In years of widespread overflows on this

delta, the crops on this area of 9 000 sq. miles will be a total loss to the nation, increasing the above estimate of $376 320 000 by the loss of the crops on that 5 700 000 acres of cultivated land.

The annual loss to the United States appears to be $370 000 000 in a small area of its territory, for want of protection against the overflow of this river.

As an indication that the writer's views of the value to the nation of this small fraction of its area are within conservative lines, an extract is given from the conclusion of an address before the Commercial Club of Cincinnati, in December, 1903, by B. M. Harrod, Past-President, Am. Soc. C. E., one of the oldest and ablest members of the Mississippi River Commission, bearing directly on this subject:

"Now, this is the proposition: There is an area of 20 000 000 acres of the most fertile land in the world. At least three-quarters of it are susceptible of the highest cultivation. Its potential products are diversified; including wheat, corn, cotton, sugar and rice. The timber wealth of the valley is immense, and it is in every way favorable from climate, soil and means of transportation for development by the farmer, the manufacturer, and for railroads. Without levees it is a jungle; an uninhabitable swamp. With levees each and every acre can be protected at a cost not exceeding three dollars, and this protection can be maintained at an annual cost of ten cents per acre. Nearly two-thirds of the work is now done."

The prospective addition of 4 000 000 bales of cotton and a proportionate yield of sugar—both among the necessities of life—to the resources of the country, renders the subject one of National importance. It seems to be a matter of paramount importance, and demands an investigation as to the causes of this condition of things, and to ascertain if means cannot be suggested that will, at least, mitigate the effects of these periodical disasters.

The magnitude of this problem should suggest diffidence in approaching its solution. In the past, it has engaged some of the best and most experienced engineers of the country; whereas they have worked along proper and conservative lines, the means at their command have been entirely inadequate for carrying out their plans. At the beginning of 1861, the levee systems below Memphis were in good condition, as far as constructed. The ravages of war, however, and the unrestrained power of the river, exerted in some very high overflows, swept off almost the last vestige of the levees existing at that time.

Since the close of the conflict (1865) the States bordering on this river have made remarkable progress in the rebuilding of the levee system, until at the present time, there is substantially a continuous system of levees on each bank from Cape Girardeau to a point below New Orleans. Heretofore, each State has controlled its own levee system, but with means entirely inadequate to the magnitude of the work. It is believed there should be one controlling head for the entire levee system of the delta, and there should be ample means to carry out its plans. It is believed, moreover, that in a few years Congress will recognize the national importance of this work in connection with the plans for the deep-water navigation of this great inland sea, and provide ample means for carrying out the plans determined on by this central control.

The writer has had especial opportunities for the study of the conditions at both ends, as he conceives it, of this great problem, and desires to record, in a tentative way, the views and opinions that have impressed themselves upon him. From 1856 to 1861, as a civil engineer, he had charge of the construction of the Memphis and Ohio, one of the principal railroads leading from Memphis, Tenn. Subsequently, 1867 to 1873, after the ravages incident to the war, he had charge of the reconstruction of the same road. During that period he witnessed a great many of the floods which devastated the delta. In 1866 he was a cotton planter in the Yazoo Basin, and was a sufferer from the flood of that year. During all this time, with the predilections of a civil engineer, he naturally studied all questions bearing on the causes of these floods. He sought information from all accessible sources; from old and experienced river men—the captains and pilots of steamboats—from cotton planters, and from civil engineers.

During these periods of overflow, this delta became an inland sea, 40 miles wide opposite Memphis and from 70 to 80 miles wide opposite the Yazoo Basin, with an occasional island where spots of dry land on the plantations were in evidence. In the river channel, its waters were extremely muddy—angry-looking, awe-inspiring and repulsive. This indicates the extent of the writer's opportunities for forming correct opinions at the lower end of this problem.

It happened that in 1876 circumstances drifted him to the Northwest, where 28 years of his life were spent in Montana and Wyoming.

Seven or eight months of each year for the first eight years were spent in the mountains at the heads of the various tributaries of the Missouri and Columbia Rivers, whence come the melted snows that make up principally the "June rise" that occasionally has such a potent influence in the devastation of the Great Delta.

Twenty years were afterward spent on a cattle ranch on the Upper Grey Bull River, at an elevation of nearly 7 000 ft. above tide. Within ten miles to the south is Franc's Peak, the elevation of which is given by the Geological Survey as 13 300 ft. above sea. On each side of the valley are mountain ranges with elevations of 10 000 and 11 000 ft.

During those twenty years' residence in this locality, a daily record was kept of temperature and precipitation, especially of snowfall. The necessity for irrigation called attention to all conditions influencing the water supply. The minimum temperature during the winter months varied from 24° to 47½° below zero. The annual snowfall varied from 50 to 110 in., the latter being unusual and extreme. The average snowfall per year was from 75 to 80 in., the average annual precipitation being about 13 in.

The conditions obtaining in this locality, as regards temperature, snowfall, and snow melting, may be safely taken as those on all the tributaries of the Missouri River of the same altitude. This altitude is about the upper limit of even forage crops. Basing opinions and views on the observations and knowledge gained by a long residence on the Lower Mississippi, and on a longer residence at the upper end of this great problem, among the mountains at the headwaters of the various tributaries of the Missouri, the following condensed statement gives the results of this study, premising that it applies more especially to conditions as to the delta preceding the year 1861. During the "War between the States," the levees built previous to that date were destroyed to a greater or less extent, by military acts or by destructive floods.

Each year there were two periods of flood-waters in the Mississippi River below the mouth of the Ohio. The first flood, styled the "spring rise," came almost entirely from the Ohio River, reinforced to some extent by the melted snows and rains from the Lower Missouri and the Upper Mississippi proper. The advent of this flood at its mouth varied in time, volume, and intensity with the climatic conditions of each spring.

The second flood, styled the "June rise," came mostly from the Missouri River, caused, as a governing factor, by the melting of the accumulated snows of winter at the sources of its larger tributaries having their origin in the great Continental Divide.

The advent of this flood at Cairo, and its volume and intensity, were dependent mostly on conditions at its head; the amount of snowfall during the previous winter on its western water-shed, more especially in the vast pine forests; the time, whether early or late, of the first warm spells of spring, their duration and intensity (it required a week or often ten days of warmth sufficient to melt snow at night, at an elevation of 8 000 ft., to produce the highest floods in the local streams); and, to some extent, on the local rains in the lower water-shed of the Missouri and Upper Mississippi Rivers. The spring floods from the Arkansas and Red Rivers, having their origin in the same range of mountains, but much farther south, did not contribute so seriously to the disastrous effects in the lower delta, because they usually came earlier, and had passed down before the advent of the "June rise" from the Missouri.

In a majority of years the crest of the "spring rise" had passed Cairo before the advent of the "June rise" from the Missouri, in which event it passed down to the Gulf with but little damage to plantations or levees. In cases where cultivated fields were overflowed, the water subsided in time to plant cotton and corn and to raise a partial crop of each.

Following in the wake of the "spring rise" came the "June rise," but, as the former had substantially passed out of reach, the latter passed down to the Gulf without doing material damage to either levees or plantations.

As is evident from this statement of conditions, if, from various climatic causes, the "June rise" from the Missouri came tumbling down upon the "spring rise" before the latter had passed out of the way, there would be one of those disastrous overflows which the system of levees built previous to that date had never withstood, resulting in enormous losses to the plantations, usually, in the entire delta. Such floods were so late in subsiding from the cultivated fields—sometimes as late as August 1st—as to prevent the raising of either cotton or corn for that season. The best information was that these disastrous floods came in cycles of about seven years, and varied in intensity, in accordance with the volume of water brought down in each of the two

"rises," and, in a large measure, as to the point at or near Cairo, at which the two floods merged.

Since the writer's personal knowledge of conditions in this delta up to 1873, more than thirty years ago, these conditions have been modified to a certain extent. The "spring rise" from the Ohio River water-shed makes its advent below Cairo earlier and in larger volume than in former years, in a volume so much larger that the levees are often crevassed with the sequence of great damage to the planting interests. Although this "spring rise" may have passed down and out of the way of the "June rise," yet, if the latter was of an average, or greater than the average, volume, its waters would overflow through the "crevasses" made by the earlier rise, widening them and resulting in about the same amount of damage as would have occurred had the conditions been the same as they were previous to 1861.

Notwithstanding the somewhat earlier advent of the "spring rise" at Cairo, it has not prevented, at times, that conjunction of events which occurred in former years, when the two "rises" combined their floods there, with such disastrous effect in the delta below. The only perceptible difference is that these combinations of the two "rises" occur at somewhat longer intervals than seven years, as the writer is informed.

The much earlier advent of the "spring rise" at Cairo can be attributed to but one cause, namely, the denuding of its entire water-shed of the timber and underbrush which covered it thirty or forty years ago. The result, as was to be expected, is that the melted snows and rains are precipitated into its valleys much earlier and in larger volume, resulting in great destruction of property, and in hardships to the people of Pittsburg and the entire Ohio Valley below.

The destructive effects of the "spring rise" on the delta below are caused when the early floods from the Cumberland and Tennessee Rivers merge near its mouth with the floods from the upper tributaries of the Ohio.

This condition of affairs in the Ohio Valley is an object lesson as to that which will occur on a much larger scale in the great delta, unless we not only preserve, but add to, the extensive forests now existing at the sources of the principal tributaries of the Missouri, which furnish such a large percentage of the annual floods poured into it at Cairo.

Imagine for a moment that the forests on the eastern slope of the great Continental Divide could be suddenly wiped from the face of the mountain, and that their winter snows were exposed to the direct rays of the sun in spring and summer; there would be such an early melting of the snows in the spring that the "June rise" from the Missouri would make its advent near Cairo so early every year that it would merge with the "spring rise" from the Ohio, overtop the levee system, and create such devastation that in a few seasons it would depopulate the entire delta, and New Orleans would become a thing of the past. This is not a fancy sketch. The same causes, however unrestrained, that have produced the conditions before alluded to on the Ohio River water-shed (the greed of Man), combined with the ravages of forest fires, which, when under full headway are as uncontrollable as the floods of the Mississippi, render this catastrophe not impossible. Fortunately for the country, this is not a supposable case at this time.

President Roosevelt, soon after assuming office, aroused public opinion as to the value of these forests for irrigation, so that Congress quickly responded in liberal appropriations, the Forestry Service was at once organized along liberal lines and under the direction of Gifford Pinchot, Assoc. Am. Soc. C. E., and everything points to a thorough oversight of this most valuable asset.

A brief study of the foregoing facts and views seems to point to a simple and evident solution of the problem set forth in the beginning of this paper, that is, to control the "spring rise" from the Ohio or the "June rise" from the Missouri so as to prevent their conjunction or merging near the mouth of the Ohio, which has always produced great destruction below.

Could this desirable object be effected, a system of levees in the delta adequate to take care of the "spring rise" from the Ohio would substantially be sufficient to take care of the "June rise" which, though of greater volume, would flatten out when not encountering any back flow from the Ohio flood.

There does not appear to be much relief in sight in reference to the control of the "spring rise" from the Ohio. Reforesting its water-shed would give relief to its immediate valley, but none below its mouth. On the contrary, the earlier advent of that flood at its mouth gives it the more time to get out of the way of the "June rise." The proposal

to store the flood-waters of this stream in immense reservoirs, by impounding its waters behind dams, does not impress the writer as either feasible or practicable. The cross-section of its valley below Pittsburg is too flat for reservoirs of large capacity, and, above that point, the valleys of its tributaries are too narrow and their gradients too steep to be available for that purpose. The same conditions exist as to its other important tributaries, the Cumberland and Tennessee Rivers.

For the past ten years the General Government has been engaged in building an extensive system of reservoirs for the purpose of impounding the spring and summer floods at the head of the Mississippi River, with the object of turning them loose in the fall to assist in the navigation of this river below St. Paul. While these works answer and will continue to answer the purpose of their construction, the volume of water impounded will not give material relief at Cairo.

It is, then, to the Missouri River that we must look for some means of holding back or retarding the advent of the "June rise," so as to give the "spring rise" sufficient time to get out of its way.

The building of artificial dams to impound these floods seems to be neither feasible nor practicable, within reasonable cost, as was contended regarding the Ohio River floods. The writer recalls sites for such dams on most of the large tributaries of the Missouri (the upper ends of cañons, with the valley of the stream above opening out into a comparatively broad valley), where, by building dams 500 ft. long, much of the flood-water of each stream could be held back. Without going into the calculations of the cost of such works on all the large tributaries, it would appear to be so enormous, when compared with the cost of the means suggested by Nature, and spread out all around these proposed dam sites, that comparison of the merits of each scheme would seem to be unnecessary.

On the head-waters of its principal streams, Nature has pointed out the means to be used in accomplishing this desirable end by providing those dense pine forests that serve as vast storage reservoirs for the accumulated deep snows of winter. These forests, by their shade, protect the snows from the direct rays of the sun, allowing them to melt so gradually as to prevent damage in the streams below. It is the rapid melting of the snow on the large areas on and contiguous to the Continental Divide, which from various causes have been denuded

of their forests, that causes flood damage in the streams below, and is the main factor of the "June rise."

The lead pointed out by Nature must be followed. The present area of these forests must not only be preserved, but must be added to, by reforesting the large areas of open or prairie land which are surrounded by or are contiguous to the main bodies of forested land.

On each slope of the Continental Divide, in Montana and Wyoming, and on the mountain spurs branching out therefrom, are vast pine forests extending from 30 to 60 miles in width on each side of the summit. There are also groups of mountains of considerable extent arising from the plains on each side and within 100 miles of this Divide (such as the Big Horn Range and the Teton groups), which are covered more or less with similar forests.

The limit of the timber line in that latitude is about 9 200 ft. above sea level. Below that line and surrounded by or contiguous to these forests are open or prairie lands about equal in area to that of the pine forests, which have been denuded of timber, by fire and other causes, the conditions of soil and moisture being similar to those of the dense forests adjoining them.

On these upper mountain plateaus are to be found many never-failing springs, and these, during the dry months of the fall, will serve to irrigate the young pine shoots used in reforesting. The adjoining timber will furnish all the small pine saplings needed.

Above the timber line and on each side of the Continental Divide and the mountain spurs branching out therefrom are large areas of open land, but whether it has been denuded of its timber by fires, or whether its condition is normal and due to the frigidity of the climate in winter, is not apparent. On the eastern slopes there is generally a sufficient depth of good soil which has been drifted from the western slope by the high westerly winds prevailing at all times.

During the storms of winter, there form on the eastern slope snow-drifts deep enough to last very often until snow flies again. The writer is not sufficiently informed as to the principles of forestry to give an opinion as to whether these open areas above the timber line can be reforested; but there are large areas with sufficient soil, and water from the snow banks, for that purpose, if the winter temperature does not preclude the idea.

There are no records in existence as to the present extent of the

pine forests on the head-waters of the Missouri River tributaries, nor of the open areas now denuded of forests from unknown causes, lying among or contiguous to the existing forests below the timber line.

It is believed, however (and this belief is based on a somewhat intimate knowledge of about 100 miles length of this Continental Divide and its numerous spurs), that below the timber line there are areas of open land which can be reforested, and that these are sufficient to equal in extent the areas of existing forests.

As before stated, the record of snowfall at the Four Bear Cattle Ranch, on the Grey Bull River, for a term of years, averaged about 80 in., and its climate could be taken as the average of conditions at points of similar elevation (7 000 ft.) around the water-shed of the Missouri River. It is thought that the snowfall on the high mountain plateaus is greater than in the valleys below, but there are no data at hand to sustain this opinion.

The snowfall, as measured each day, was of light weight, the 80 in. of average snowfall representing about 8 in. of water.

This 80-in. depth of snow has fallen between September 1st and May 1st, and, in the pine forests, at altitudes of from 7 500 to 8 000 ft., and represents all the rain and snowfall between these dates. At that altitude freezing temperature is one of the principal preservatives of snow, especially in the warm months. There are few nights at that season, among the pine forests, when snow does not "crust over" sufficiently to support the weight of a man or even a horse.

In estimating the amount of precipitation that goes to make up the floods of the Missouri River, there must be added to this 80 in. of snow the precipitation in May, June, and July of each year. This represents the total annual precipitation for the year, except that which occurs in August. As the record for August around this water-shed is only about $\frac{65}{100}$ in., it can be safely assumed that this annual precipitation of that altitude represents the stored up precipitation for each year in those pine forest reservoirs.

As will be hereafter explained, one-half of this precipitation (say, $6\frac{1}{2}$ in.) represents the portion covering the open areas, surrounded by or contiguous to the forests, which turns into water in May or during the first days of June, and flows into the larger tributaries within 12 hours after melting, thus forming the controlling factor of the famous "June rise." The volume and intensity of this flood are caused, not

only by the volume of water thrown into the stream within the space of a month, but by the melted snow being at once tumbled down the steep mountain into the water channels below in an incredibly short space of time, thus adding to the intensity of the flood.

As contributory to the "June rise" may be mentioned the melting of the deep snowdrifts of the great plains west of the 100th meridian and the local rains of the Lower Missouri and Upper Mississippi.

Contributory to the "spring rise," to some extent, are the melted snows of the open areas of these mountains, that lie above timber line (9 200 ft.). As these snows lie usually in immense drifts, they do not melt much before the latter part of July and August, too late to affect the "June rise."

The volume and intensity of the "June rise" are dependent to a certain extent on the coincidence or conjunction of all these factors. The melted snows from the forest reservoirs are, undoubtedly, the controlling factor.

In April, sometimes earlier, under the influence of the "Chinook" or warm winds from the Pacific, the snows covering the plains below the foothills of the Continental Divide commence melting, causing the first floods of the main stream and tributaries, which soon completely sweep out the ice, often creating immense gorges down as low as Omaha.

The writer's information is that the influences of these Chinook winds at its head, frees the Missouri of ice much sooner than is the case with the Upper Mississippi.

By April 1st, the snows in the timber on the high mountain plateaus have usually settled to a depth of from 4 to 5 ft. On account of the low temperatures peculiar to an altitude of 8 000 or 9 000 ft., the snow does not usually commence melting in the open tracts until about May 1st. It melts so rapidly, however, that by June 10th the grass is in such an advanced stage that cattle from the ranches below are taken, at about that date, to these upper mountain plateaus to be kept there until the November snows.

In the forests contiguous to these open spaces, the snow does not usually disappear until the beginning of August.

This, in a nutshell, tells the value of forests in conserving the snows of winter. In other words, forests of average density and at the average altitude of their habitat, 8 000 to 9 000 ft., will preserve

the snows of winter from melting two months longer than open areas exposed to the direct rays of the sun.

As an object lesson in support of this, the writer cites his experience on the Grey Bull, on July 16th, 1880, in crossing the Continental Divide from the waters of Snake River opposite the Upper Geyser Basin of the Yellowstone National Park. The Divide is comparatively flat, has an elevation of about 8 200 ft., with about an average density of forests, and at that date was covered with a depth of from 3 to 6 ft. of snow, depending somewhat on its exposure to sunshine at the small openings in the timber. The area of these openings ranged from a few acres to 20 acres. There may have been some drifting of snow in places. As is always the case at this altitude, the snow banks are crusted over during the night from cold (one of the elements of snow preservation at this altitude), and the party was compelled to take advantage of it, and get the pack animals across the snowdrifts on the summit early in the morning. The small openings in the forests, before alluded to, had been freed from snow so long that grass was so far advanced in growth as to afford enough pickings for nooning for the horses. The previous winter had been an average one for snowfall, and that summer an average for heat. This experience afforded an admirable object lesson as to the efficacy of forests for preserving snow.

Now apply these facts and conditions to the entire Missouri River water-shed of like altitude, 7 000 ft. and above, to the Milk, the Sun, the Jefferson, the Madison, the Gallatin, the Yellowstone, the Big Horn and the Platte Rivers. As at present, one-half of the winter's snows that cover the open spaces will pass off in May or thereabouts, and will not be of benefit for local irrigation on the smaller streams, for it is not then needed, nor on the larger streams below for navigation, for local rains have already provided a sufficiency.

Now conceive the entire water-shed of this mighty river (below the timber line and well above the agricultural belt, say 7 700 ft.) to be clothed with forests of average density. When spring comes, the snow will commence melting in that belt about May 1st, and instead of half the winter's snowfall melting during that month, and causing as a main factor the "June rise," it will melt gradually, and will not have disappeared before August 15th. Such gradual melting will not create those spasmodic floods, common in the local streams below, that

often do so much damage to irrigation work. The floods in all the large tributaries are held back by these causes, and, in the outcome, the "June rise," about which so much has been said, will reach Cairo on an average one month later than usual. If the levees of the delta are of average strength, they will have taken care of the "spring rise" without any serious damage having been done. It will have passed down and out of the way of the "June rise," held back from causes before described, and the latter rise, from the same causes, will be of much less intensity than in former years, and will pass down to the Gulf and do but little damage.

The "June rise" is not caused entirely by melted snows, but is reinforced to some extent by local rains on the Lower Missouri and the Mississippi. The retarding of the melting snows of the "June rise" one month, by the means before described, will result in lessening its intensity to that amount, as the effects of the local rains will have passed down.

Reforesting the upper water-shed of the Missouri, as pointed out, is a work more expensive in the time required in its accomplishment than its cost in dollars and cents.

No plan has been suggested for the control of the floods of this mighty river which will not require years of time, patience, and labor. It is believed that the plans laid out by the Mississippi River Commission, as understood, are along conservative lines, as regards this great delta, that is, to hold the river to its present channel by the revetment of its concave or caving banks, by contracting its channel at bars by proper works, and, where necessary, by dredging these bars for navigation purposes. In addition to these works is considered (as the writer understands the views of the Commission) a substantially built system of levees on each bank. To carry out these plans will require much time and patience, with proportionately much labor and expense.

To make a permanent success of these works in the delta, it is considered essential that the high floods of the Ohio and Missouri water-sheds shall be kept apart. The only feasible means of accomplishing that desirable object is by impounding the flood-waters of the latter stream in the immense forest reservoirs at its head.

Reforesting the open areas among these forests, will be small in expense compared to the work in the delta. For a few years it will

require the labor of as many men as can be worked to advantage for five months of each year, in resetting the pine saplings, and in making the small ditches necessary for irrigating them. This will require about 1 000 men for a few years. After the shoots have taken root, fewer men, who should be expert at irrigation, will be required. Nature, with the warm rays of the sun, will do the rest. It is believed that within five years after this work is started its beneficial effects will be apparent.

To make a success of this reforesting will require the application of the laws of forestry suitable for each latitude of the North American Continent and to each species of pine found most suitable; and this must be combined with the experience of expert irrigators and much patience and care.

When it is considered that the successful accomplishment of this reforesting is one of the main factors, in conjunction with the contemplated works in the delta below, in adding to the resources of the Nation each year from $300 000 000 to $400 000 000, as estimated heretofore in a detailed report, it becomes a matter of national importance, and should be carried through regardless of whatever expense, within reasonable bounds, its accomplishment may require.

Closely allied to, and as one of the most important outgrowths of, forest preservation is the Reclamation Service, brought to the front by the broad-minded statesmanship of President Roosevelt and at once adopted by Congress. Its design is to reclaim, under a wisely guarded law, the arid regions of the mountain States on each slope of the Continental Divide, by expending the proceeds of the sales of the public lands in the construction of reservoirs and ditches leading therefrom for purposes of irrigation. One of the most direct benefits to the Nation from the construction of this reclamation system, when finished, will be the large annual saving in the production of the sugar beet, now so quickly and profitably changed into sugar, one of the necessaries of life. The belt of country lying between the 100th meridian and the Continental Divide and between New Mexico and the Canadian line, is eminently adapted to the production of the sugar beet. It is believed that the portion of this belt, under the influence of the Reclamation Service, on each slope of the Great Divide, when brought under proper cultivation, can produce enough sugar, in addition to what is now being produced, to take the place of the 2 500 000 tons now imported from foreign countries.

In other words, these millions of dollars now being spent on the Reclamation Service will add each year to the resources of the Nation, when completed, at least $200 000 000, besides other benefits of great value which will naturally suggest themselves.

Bearing on the question of its influence in holding back the flood-waters of the streams making up the great "June rise," A. P. Davis, M. Am. Soc. C. E., Chief Engineer of the Reclamation Service, has kindly furnished the following table of the "Irrigating Projects" under construction in the Missouri River drainage basin:

Projects.	Acreage.	Discharge, in second-feet.	Storage, in acre-feet.
Huntley, Mont.	33 000	400	.....
Sun River, Mont.	300 000	3 500	600 000
North Platte, Nebr.	300 000	3 500	1 000 000
Lower Yellowstone.	67 000	800	.....
Milk River, Mont.	80 000	1 000	.....
Williston, N. Dak.	40 000	500	.....
Belle Fourche, S. Dak.	100 000	1 200	200 000
Shoshone, Wyo.	150 000	1 800	400 000
Totals.	1 070 000	12 700	2 200 000

The experience derived from irrigation on a small scale on the local streams of that water-shed is that the water which goes through the ditches and is turned out from them, is soaked up by the soil and does not return to the stream for a month or more. This happens in cases where the ditches are not more than $\frac{1}{4}$ mile from the stream. In the case of the above-named projects the main canals will be from 1 to 3 miles distant from the streams from which they flow. It can be estimated safely that the water passing out of the main canals of the above "projects" during May, June, and July, does not return to the streams from which they radiate for such a length of time that this discharge, together with the water impounded by the dams, can be considered as reducing the volume of the "June rise" to that extent. The volume of water thus held back amounts, by calculation, to about 4 500 000 acre-ft.; not a large volume, yet there are times when it might have a perceptible effect.

It must be borne in mind that a test has never yet been made as to the ability of the two earthen walls, comprising the levee system on each bank of the Mississippi between points opposite Cairo and to and

below New Orleans, to withstand the pressure that would be brought to bear, did the "spring rise" from the Ohio and the "June rise" from the Missouri combine near Cairo and attempt the passage to the Gulf. Judging the future by the past, the result, with the existing system of levees, would at least be problematical.

Neither the magnitude of the problem of the successful control of this river nor the benefits to be derived therefrom, are fully realized by the country. The history of engineering affords no precedent to serve as a guide in solving the problem. At flood tide its forces are irresistible when brought in direct antagonism to the puny work of Man. At times its actions appear almost as erratic as a child's. It has no respect for State's rights. In a few weeks time it will slice off a large area from one State and as quickly deposit it below on the opposite bank, to add to the territory of a neighboring State. It is only by humoring its eccentricities and guiding its inherent power, that Man can accomplish the solution of its control. This plan was successfully carried through at its mouth, when it was forced to dig out a channel 35 ft. deep (where a depth of 15 ft. had existed) across sand-bars formed by its own currents, by the construction of the jetties.

The cost of the work necessary for the control of the flood-waters will be great. The benefit to the United States arising therefrom by adding to its resources 4 000 000 bales of cotton and a proportionate amount of sugar each year makes it evident that its cost should be borne by the Nation and not by the comparatively few cotton and sugar planters of the delta, as has heretofore been done through State agencies. It should be evident that it is as much the province of the General Government to take care of the harbors of this vast inland sea and the channels leading to them as to take care of the harbors and channels of the seaboard and the Lake States.

As a means of keeping open this important channel of commerce, the building of levees, the revetment of the caving banks on their front, the contraction of its channel at low water in the interests of navigation, are the same means rendered necessary in the construction of the same works for the purpose of prevention of disastrous overflows. Any plan of river improvement for navigation without the aid of levees on the banks, would be a serious mistake and would result in failure. A bad crevasse in the levees would decrease the volume of water below, and counteract one of the governing ideas in the plans of

the Mississippi River Commission, viz., the contraction of the currents for scouring purposes.

The foregoing points out an additional reason why the cost of this great work should be borne by the Government. It carries with it the result that this entire system of work—the control of overflows by levees, and the works for the improvement of navigation—shall be under one central control. The two systems go hand-in-hand, and are interdependent.

To recapitulate: The solution of this great problem consists in keeping apart the "June rise" and the "spring rise," so that the latter floods will have passed before the advent of the former at the mouth of the Ohio.

As noted before, there is no relief to be expected from the Ohio. It is left, then, to the Missouri water-shed to furnish the means for the object required. The head-waters of this stream must be impounded in immense reservoirs, for such a length of time as will without doubt prevent the "June rise" from making its advent until the "spring rise" has passed.

Nature has constructed, as an object lesson, those immense storage reservoirs for protecting the winter's snows in the dense forests in and near the great Continental Divide. It directs us to reforest those large areas, which at one time were clothed with dense forests, but which have been denuded by fire and other unknown causes.

Nature teaches us that these open spaces, once clothed with forests, can be reclothed with similar forests if Man will use the means indicated by Science. Then will follow the slow melting of the conserved winter snows, thus keeping apart the "June rise" and the "spring rise," and, in conjunction with a substantial system of levees in the delta, this will afford the only solution for the permanent control of the periodical floods that sweep down through this great delta.

In the preparation of this paper, the writer is indebted to H. N. Pharr and W. I. Hardee, Members, Am. Soc. C. E., for information as to conditions in the Mississippi Delta subsequent to 1873, and to A. P. Davis, M. Am. Soc. C. E., of the Interior Department, and to Messrs. Kellogg, Bonsteel, Plummer, and Zoo, of the Agricultural Department, for valuable statistics.

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PAPERS AND DISCUSSIONS

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ELECTRIC RAILWAYS IN THE OHIO VALLEY BETWEEN STEUBENVILLE, OHIO, AND VANPORT, PENNSYLVANIA.

By George B. Francis, M. Am. Soc. C. E.

To be Presented January 6th, 1909.

There has been constructed during the past two years, along the northerly and westerly banks of the Ohio River, between Vanport, Pa., and Steubenville, Ohio, about 40 miles of double-track, standard-gauge, electric railroad, which affords this busy, thriving, industrial section a high-grade interurban road.

The lines form the connecting links uniting the East Liverpool, Wellsville, Steubenville, and Toronto systems, and, with their connections, at Vanport on the north and Steubenville on the south, make possible through travel by trolley between Pittsburg, Pa., and Wheeling, W. Va., except for a short piece of track, now under construction, in the vicinity of Sewickley.

The configuration of the country through which the road has been built is such as practically to preclude the location and construction of a future competing line.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

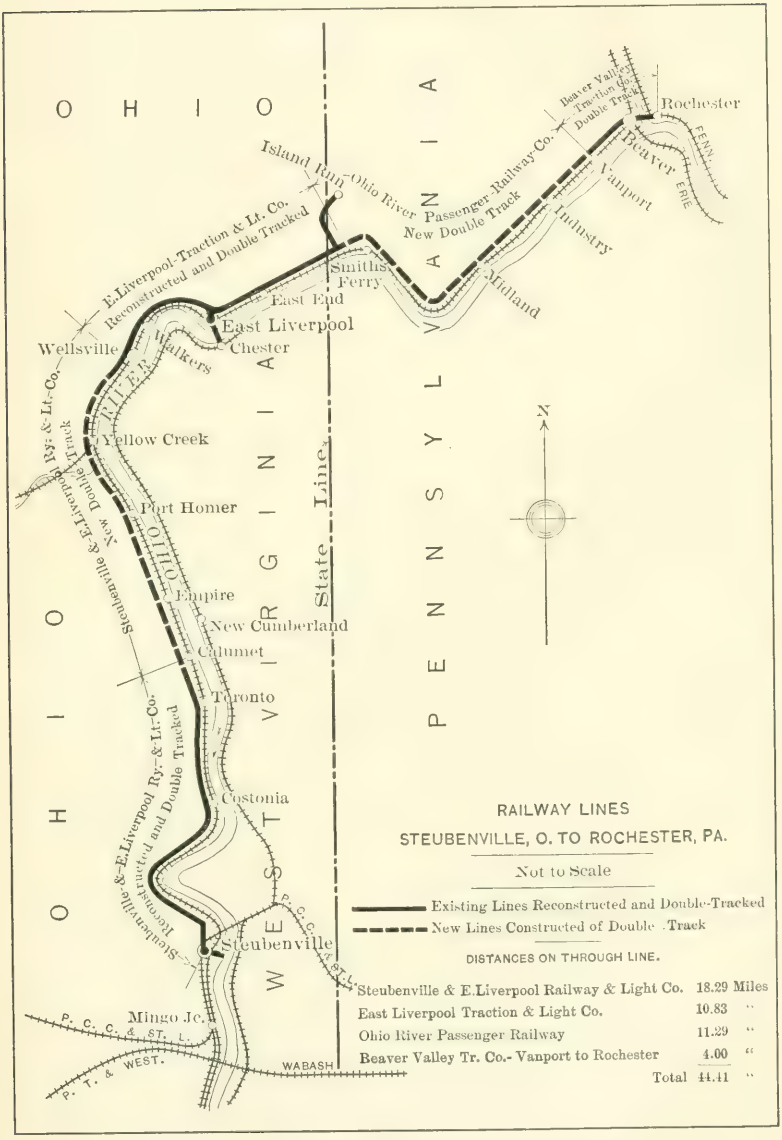

OHIO
PENNSYLVANIA
WEST VIRGINIA
OHIO
Island Run
Ohio River Passenger Railway Co.
New Double Track
E. Liverpool Traction & Lt. Co.
Reconstructed and Double Tracked
Beaver Valley Traction Co.
Double Tracked
Rochester
Beaver
Vanport
Industry
Midland
Smiths Ferry
East End
East Liverpool
Chester
Walkers
Wellsville
Yellow Creek
Port Homer
Empire
New Cumberland
Calumet
Toronto
Costonia
Steubenville
Mingo Jc.
Wabash
P. C. C. & St. L.
P. T. & West.
State Line
Ohio River
Steubenville & E. Liverpool Ry. & Lt. Co.
New Double Track
Steubenville & E. Liverpool Ry. & Lt. Co.
Reconstructed and Double Tracked
N
RAILWAY LINES
STEUBENVILLE, O. TO ROCHESTER, PA.
Not to Scale
Existing Lines Reconstructed and Double-Tracked
New Lines Constructed of Double Track
DISTANCES ON THROUGH LINE.
Steubenville & E. Liverpool Railway & Light Co. 18.29 Miles
East Liverpool Traction & Light Co. 10.83 "
Ohio River Passenger Railway 11.29 "
Beaver Valley Tr. Co.- Vanport to Rochester 4.00 "
Total 44.41 "

FIG. 1.

The railways described have been built by three constituent companies:

The Steubenville and East Liverpool Railway and Light Company.—This company owns and operates the properties of the Steubenville Traction and Light Company and the Toronto Electric Light and Power Company. The former of these acquired properties owned and operated the street railway system in Steubenville, and a single-track line to Toronto on the north, a distance of about 10 miles. The new company has reconstructed this single-track road, improving its alignment and grade at various places by locating the tracks on a private right of way; has added a new second track between Steubenville and Toronto, and has built a new double-track road between Toronto and Wellsville, a distance of 7.86 miles. The company also furnishes light and power for commercial purposes in Steubenville and Toronto. At Steubenville, the southerly terminus of the system, connection is made with the Wheeling Electric Railway, which reaches Wheeling, W. Va., a distance of about 21 miles, also with the Steubenville and Brilliant Line, on the west side of the river.

The East Liverpool Traction and Light Company.—This company serves the East Liverpool district, which extends eastward as far as the State line between Ohio and Pennsylvania, and southward to the southerly limits of Wellsville, and includes the street railway system in East Liverpool, a branch line across the river to Chester, W. Va., and a 3-mile spur track to the company's coal mine up Island Run. It also furnishes light and power for commercial purposes in East Liverpool, Chester, and Wellsville. The initial move in improving and increasing the transportation facilities demanded by this territory was made by this company. The road was originally a single-track line, lying wholly in the highways and streets, and had several dangerous steam railroad crossings at grade. In the reconstruction of that portion of the system which forms a part of the through main line, provision was made for a double track, which has been laid partly in new location, and as far as practicable on private right of way, improving the grade and alignment, and eliminating the grade crossings.

The major portion of this property was reconstructed during 1905 and 1906, thus preceding the construction of the stretches of road southward to Toronto and Steubenville, and northward from East Liverpool to Vanport, which has resulted in the continuous line of about 40½ miles of double-track road described herein.

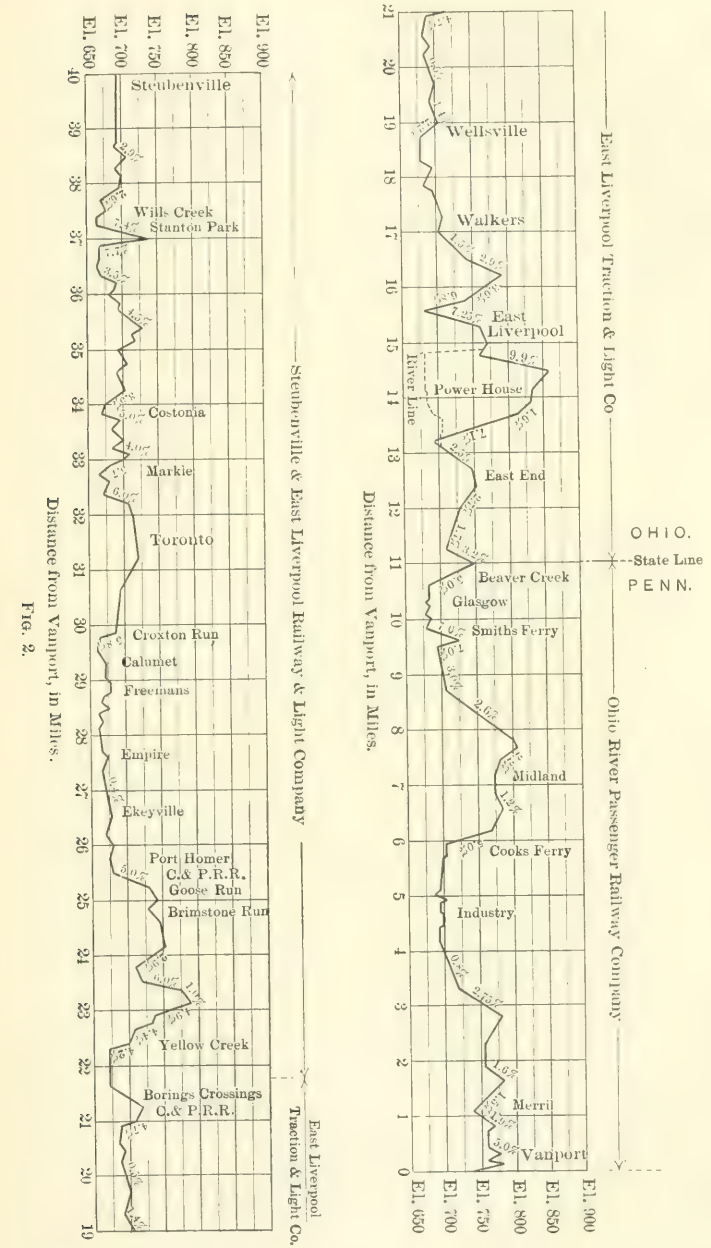

PROFILE OF GRADE LINE BETWEEN VANPORT, PA. AND STEUBENVILLE, O.
East Liverpool Traction & Light Co
Ohio River Passenger Railway Company
OHIO.
State Line
PENN.
Wellsville
Walkers
East Liverpool
Power House
River Line
East End
Beaver Creek
Glasgow
Smiths Ferry
Midland
Cooks Ferry
Industry
Merril
Vanport
Distance from Vanport, in Miles.
Steubenville & East Liverpool Railway & Light Company
East Liverpool Traction & Light Co.
Steubenville
Wills Creek
Stanton Park
Costonia
Markie
Toronto
Croxton Run
Calumet
Freemans
Empire
Ekeyville
Port Homer
C.& P.R.R.
Goose Run
Brimstone Run
Yellow Creek
Borings Crossings
C.& P.R.R.
Distance from Vanport, in Miles.
El. 650
El. 700
El. 750
El. 800
El. 850
El. 900

FIG. 2.

The Ohio River Passenger Railway Company.—This company has built the new double-track road between East Liverpool (where connection is made with the lines of the East Liverpool Traction and Light Company) and Vanport, a distance of 11.29 miles. At the latter place connection is made with the lines of the Beaver Valley Traction Company, with which company a traffic agreement has been made for operating cars into Rochester, Pa. Arrangements are being perfected with the Pittsburg and Lake Erie Railroad for a connection at Beaver and for the sale of tickets for through travel to any point on either system.

A tri-party agreement has been executed with the foregoing constituent companies permitting the joint use of the main-line trackage between Steubenville and Rochester, a distance of 44½ miles.

Location.—The portion of the Ohio Valley traversed by these connecting lines is narrow, with high and, in some places, steep and precipitous hillsides and bluffs alternately advancing and receding from the shore of the river.

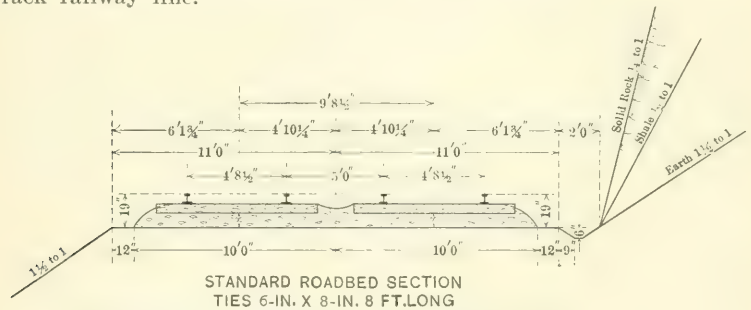
The location along the bank of the river on a low grade line was practically pre-empted by the Cleveland and Pittsburg Division of the Pennsylvania Lines, and it required considerable engineering study and skill to secure private right of way, and locate a line which would not only give easy curves and grades, but avoid expensive trestles and viaducts.

Railway Construction.—The permanent way consists of a graded roadbed, located partly in the streets and highways, but for the greater portion of the distance on a private right of way, laid with 85-lb. rails, and ballasted in a first-class manner. The alignment and grades are such as are compatible with high-grade urban and interurban roads. All streams and waterways are crossed by substantial bridges and culverts, the former being of steel and the latter of concrete or stone masonry.

Track.—The track is of standard-gauge construction, and in the main is of 85-lb. T-rail, Am. Soc. C. E. section, laid in 60-ft. lengths on 6 by 8-in. by 8-ft. hardwood ties, mostly oak, spaced 24 in. from center to center, and ballasted with gravel or broken stone. The rails are bonded for the return circuit with single 000 protected bonds, except in East Liverpool, where double bonds of the same type are used on each joint. Cross-bonds connect the rails at intervals of 1 000 ft.

The tracks are generally 9 ft. 8½ in. from center to center; but where bridges or trestles are crossed, provision is made for spacing the tracks 10 or 12 ft. from center to center. Spring frogs of standard pattern are used in all cases outside of city streets. Fig. 3 shows the standard adopted for roadway construction, outside of towns and cities, both in cuts and fills.

Grading.—Outside of cities and towns the grading was practically side-hill work, necessitating some heavy cuts and fills. In many instances the railroad right of way parallels the highway which was originally cut in the sides of these precipitous hills. This roadway was widened, providing room for both the highway and the double-track railway line.



STANDARD ROADBED SECTION
TIES 6-IN. X 8-IN. 8 FT. LONG
RAILS A.S.C.E. SEC. WEIGHT 85 LB. PER YD.-60-FT. LENGTHS.
GRAVEL BALLAST.

FIG. 3.

The work required the excavation of about 750 000 cu. yd. of material of all classes, a considerable percentage being rock, mostly shale and sandstone.

Beginning at Seventh Street, in Steubenville, the principal work may be described, in sequence toward Vanport, as follows: Grading and paving in highway for track and street widening for 2.76 miles; thence grading and culverts for double track on private right of way across the King and other property for 2.23 miles, this embracing considerable earth and rockwork; thence track grading and highway paving and widening for 1.42 miles; thence earthwork grading and bridging on private right of way through the Toronto Realty Company's property for 0.30 mile to the streets of Toronto; thence grading for track in the streets of that town for 2.02 miles; thence on private right of way and over a double-track, timber trestle at Croxton's Run, 360 ft. in length, for 0.40 mile; thence grading for track, partly in

streets, but mostly on private right of way through Calumet, Empire, and Ekeyville for 4.07 miles to the north end of the steel viaduct, at Port Homer, over the Cleveland and Pittsburg Railroad, 450 ft. long; thence side-hill grading on private right of way, including relocation of highways along what is known as The Narrows, to Yellow Creek, for 3.01 miles, this stretch including steel viaducts at Goose Run, 335 ft. long, Brimstone Run, 175 ft. long, various cattle passes, heavy retaining walls, and small bridges; thence by a single-span steel bridge, 174 ft. long, over Yellow Creek; thence under the Cleveland and Pittsburg Railroad through existing bridge openings, and side-hill grading, including the straightening and paving of the highway for 0.78 mile to Borings Crossing, a new bridge being there constructed for the Cleveland and Pittsburg Railroad over the new road and highway, thus removing an existing grade crossing; thence track grading and street paving in Wellsville, for 3.69 miles to the city line of East Liverpool, this stretch including the Wellsville steel viaduct, about 300 ft. long; thence track grading and street paving in East Liverpool for 7.14 miles to the easterly city line, this stretch including the Jethro Run steel viaduct, about 475 ft. long, the Sixth Street steel viaduct, 664 ft. long, Bradys Run culvert, Dry Run bridge, and other structures. For a considerable portion of the latter distance the highways were widened, and some of the new railroad was placed on a private right of way; thence crossing Beaver Creek by a two-span double-track steel bridge, 306 ft. long, and crossing under the Cleveland and Pittsburg Railroad through an existing opening, and in streets for 1.47 miles to a bridge constructed over the Cleveland and Pittsburg Railroad tracks, for both the new railroad and the highway, thus removing a grade crossing; thence track grading on a private right of way, and straightening, widening, and paving the highway for 2.20 miles to Midland; thence track grading and paving for 1.51 miles through this town; thence grading on a private right of way for 1.17 miles to Industry; thence track grading and street paving through this village for 0.37 mile; thence grading on private right of way (mostly side-hill work) for 4.55 miles to the end of the new double track at Vanport. This last stretch embraces the following steel viaducts and other important structures: Four Mile Run viaduct, about 323 ft. long; Barclay Run viaduct, about 275 ft. long, and Vanport trestle, about 700 ft. long.

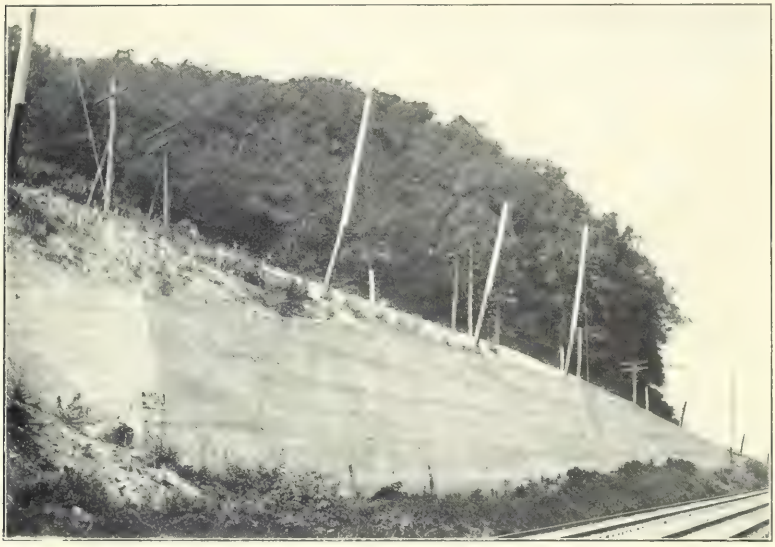


FIG. 1. RETAINING WALL, COOKS FERRY, OHIO.

FIG. 2.—YELLOW CREEK BRIDGE.

The side-hill grading was quite difficult, as a great number of slides came down the hillsides, in some instances from considerable distances, causing large expense for their removal. The clayey nature of the material, combined with water from the hills, was the cause of many of these slides. The excavated material consisted of earth, a mixture of earth and rock fragments of varying sizes, and solid rock.

The side-hill work necessitated the construction of a number of retaining walls at different points, the principal ones being at The Narrows, just south of Yellow Creek, and one near Cooks Ferry. The former is about 350 ft. long and has a maximum height of 30 ft.; each contains approximately 1 600 cu. yd. of concrete. The wall at Cooks Ferry is of similar dimensions.

Paving.—The line passes through fifteen communities, and in nearly every instance where a franchise was given for the use of streets or highways unusual requirements for paving were exacted. In some instances the entire width of the street was paved; in others, where the street grade was changed, the embankment slopes in front of residences were paved, as well as the street surface.

As this is a great brick manufacturing center, all paving consisted of vitrified brick on a sand cushion, 11 miles of track being thus paved, the cost averaging approximately $0.90 per sq. yd.

Bridges.—All important bridges, with the exception of those over Beaver and Yellow Creeks, are deck structures of the viaduct type, weighing about 700 lb. per lin. ft. of double track, and were calculated for a concentrated load of 24 tons on two axles at 10-ft. centers, or a uniformly distributed load on trusses of 1 800 lb. per lin. ft. of each track.

The standard of construction calls for 35-ft. braced towers with battered posts, and 60-ft. spans between the towers, with provision for expansion at each tower. All columns and towers are fixed to the foundations with long anchor bolts set in the piers, and at the abutments the structure is held in place by short bolts set after the erection of the steel, thus allowing for creeping during erection. The structures, though light, were built for interurban electric passenger service only, and are thoroughly braced and very rigid. The floor systems are stiffened further by carrying the ties clear across the structure. All foundations and abutments are of concrete. Fig. 4 is a typical design for these viaducts.

The bridges over Beaver and Yellow Creeks are double-track, through, two-truss structures, having rigid members with riveted connections. The load is distributed evenly over the bearings by using pin-connected shoes. At the expansion end the shoe rests on a nest of turned rollers which are free to move in a planed roller box bolted to the masonry. The structures are secured firmly to the concrete abutments and piers by long anchor bolts set in the masonry.

The Beaver Creek bridge consists of two double-track, through spans, each 153 ft. long; the Yellow Creek bridge consists of one double-track, through span, 174 ft. long. The steel for these bridges was fabricated and erected by the Fort Pitt Bridge Works.

At Sixth Street, East Liverpool, a steel viaduct was built over what is locally known as Horn Switch. It is 664 ft. 4 in. over all, is 40 ft. above the foundation at its highest point, and weighs about 233 tons. The steel for this bridge was fabricated by the American Bridge Company, and erected by the Cleveland Engineering Company.

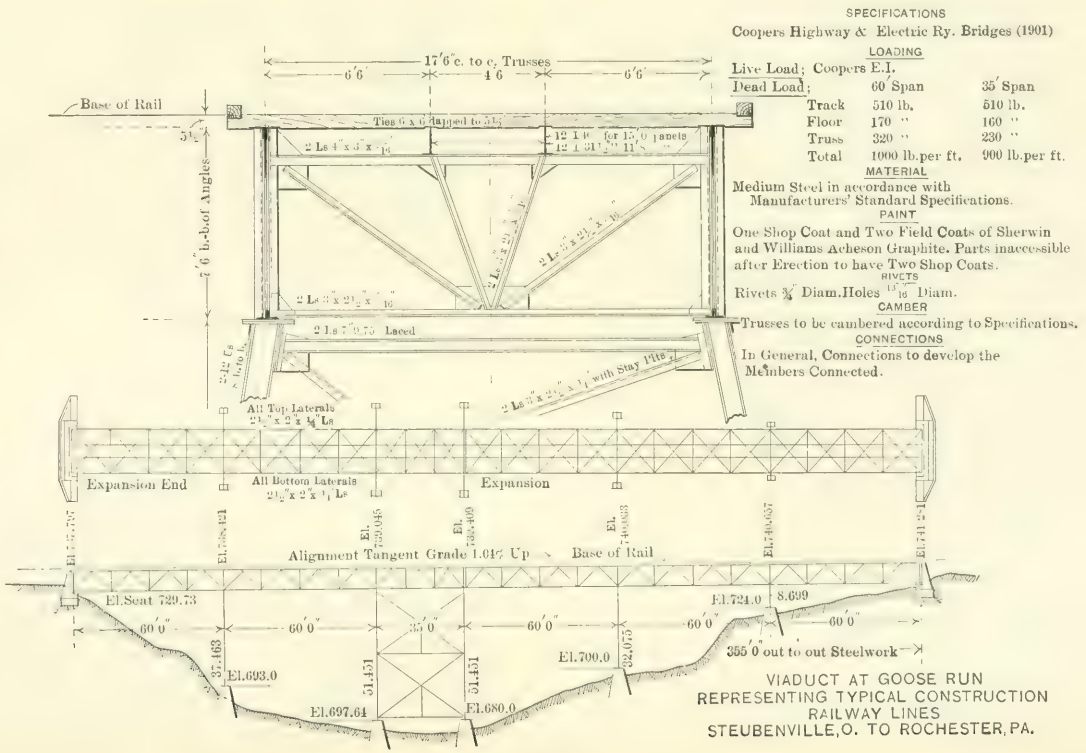
The aggregate weight of all the new truss bridges and viaducts approximates 1 100 tons.

A number of small I-beam bridges were built, and three of the existing highway structures were strengthened to take care of the loading imposed by the addition of a second track.

Power Requirements.—The power requirements of the original companies forming a part of this property were provided for in four generating stations, located at East Liverpool, Wellsville, Toronto, and Steubenville. These stations supplied current, not only for the street railway operations, but also for municipal lighting and power. To this load factor were added the requirements for the operation of about 60 miles of new track and an increase of 50% in the lighting loads at East Liverpool, Steubenville, and elsewhere.

In the changes, alterations, and additions to the generating, distributing, and transmitting system, the existing houses and equipment were utilized as far as possible.

After a careful study of the conditions, it was decided to consolidate all the generating equipment in the East Liverpool and Steubenville power-houses. The loads formerly carried by the Toronto and Wellsville stations were transferred to these main stations, and the buildings were remodeled and equipped as sub-stations, one new sub-station being built at Industry.



VIADUCT AT GOOSE RUN
REPRESENTING TYPICAL CONSTRUCTION
RAILWAY LINES
STEUBENVILLE, O. TO ROCHESTER, PA.

FIG. 4.

The difficulties of this work were increased by the necessity of keeping the generating stations in continuous operation while the changes and alterations were being effected. The work was accomplished successfully without the interruption of any service that was being rendered.

East Liverpool Power-House.—The original building was a brick structure with a wooden floor and a tar and gravel roof on timber supports. The old wooden floor was taken out and replaced with reinforced concrete, giving increased cleanliness and greatly reduced fire risk.

The load on this station was increased by the addition of that formerly carried by the Wellsville station and the new Industry substation, as well as by the increased requirements of the railroad, to provide for which the alternating-current generating equipment was increased by the addition of one 1 000-kw. turbo-generator and one 500-kw. turbo-generator, and the direct-current equipment by one new 300-kw. rotary converter and the transfer of a 300-kw. engine-type generator from the Steubenville power-house. A new 7-panel switch-board was installed to take care of this additional electrical apparatus.

The turbines as well as the alternating-current, belt-driven generators at this plant produce 60-cycle, 2-phase current at 2 200 volts. To reduce this voltage for the operation of the rotary converter, two 175-kw. step-down transformers were installed, and to raise it for transmission to Wellsville and Industry, six 175-kw. step-up transformers were provided. Excitation is taken care of by one steam- and one motor-driven exciter, either of which is of sufficient capacity to furnish excitation for all the alternating-current generators. A Tirrill regulator was installed for voltage regulation.

The boiler plant was enlarged by the addition of a wooden frame structure, sheathed with corrugated iron, in which were installed four 500-h.p. Stirling water-tube boilers. A 2 000-h.p., open feed-water heater, and two 12 by 7 by 12-in. boiler feed pumps were also provided.

In order to operate the turbines, a barometric condenser with dry vacuum pump and two motor-driven circulating pumps were installed. Water for condensing purposes is taken from the Ohio River, and, as there is about 40 ft. difference between the high and low stages of the

FIG. 1.—TRACK VIEW AT INDUSTRY, PA.

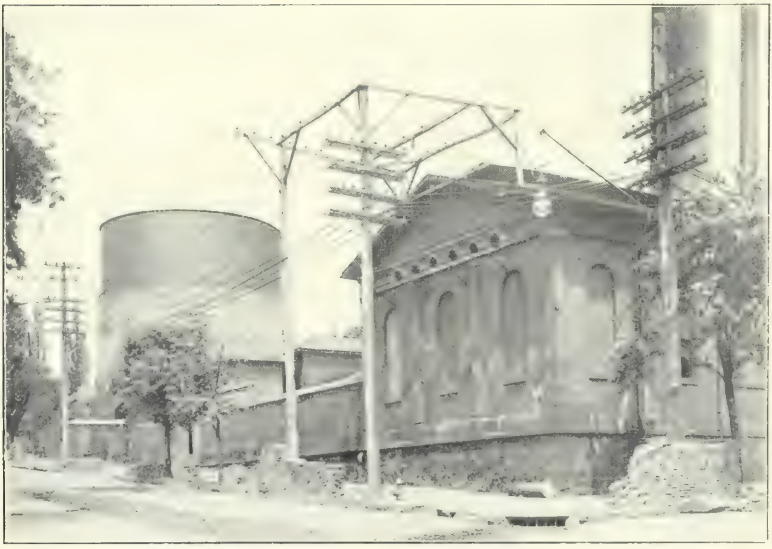


FIG. 2.—WATER-COOLING TOWER, STEUBENVILLE POWER PLANT.

river, unusual means had to be provided for elevating the water to the condensers. To this end a vertical intake or well of reinforced concrete was built on the bank of the river, extending from below the low-water to above the high-water level. Vertical centrifugal pumps were then installed at the bottom of this well and connected by vertical shafts to two 50-h.p. 2 200-volt induction motors placed above the high-water mark. From the bottom of the head-house a 24-in. intake pipe was extended to the center of the channel of the river, terminating in a timber crib.

The coal supply for this plant is obtained from the company's mine, from which it is carried over a 3-mile branch line and delivered in dump cars at the fire doors of the boilers. The generating capacity of the station has been increased from 1 200 to 3 000 kw.

Steubenville Power-House.—This is a brick structure with a brick floor and a slate roof, supported by steel trusses. A separate building, of similar construction, was enlarged to provide for the installation of two 500-h.p. boilers.

The station load was augmented considerably by the transfer of the lighting load formerly carried at the Toronto station, the increased lighting load for Steubenville, and the additional power required for the operation of the 25 miles of newly constructed railway lines which it serves. To take care of these requirements, the following additional equipment was installed: Two 500-kw. turbo-generators, producing 60-cycle, 2-phase current at 2 200 volts; two 175-kw. and two 100-kw. step-up transformers for raising the voltage to 13 200 volts, 3-phase, for transmission to the Toronto sub-station. The switch-board was enlarged by the addition of four new panels to handle this new equipment. Excitation is taken care of by one steam-driven and one motor-driven exciter, either of which is of sufficient capacity to furnish excitation for all the alternating-current generators. A Tirrill regulator was also installed.

The plant was enlarged further by the addition of two new 500-h.p. Stirling boilers, and the necessary condensing apparatus for the turbines, consisting of a barometric condenser, dry vacuum pump, and engine-driven centrifugal and circulating pumps.

As the only source of water supply for condensing purposes was the city water system, it was necessary to construct a cooling tower and an additional engine-driven centrifugal pump to deliver water

thereto. In erecting this tower, use was made of an old gas tank and well on the property.

The capacity of this station has been increased from 1 800 to 2 500 kw.

Toronto Sub-Station.—The Toronto sub-station building formerly contained the generating apparatus supplying current for the arc- and incandescent-lighting systems of Toronto, and was built of tile blocks, with a brick floor and a slate roof. As now equipped, it contains two 300-kw. rotary converters furnishing direct railway current at 585 volts, and operated from the transmission line through six 110-kw. step-down transformers, and two 110-kw. transformers which deliver 2-phase current at 2 200 volts for the incandescent-lighting circuits, and for the arc-lighting circuits by constant-current transformers and mercury arc rectifiers. The station has a 10-panel switch-board.

Current is received from the Steubenville power-station at 13 200 volts, over two 3-phase transmission lines 9¼ miles long.

Wellsville Sub-Station.—The apparatus for this sub-station is installed in a building which was formerly the Wellsville power-station, supplying Wellsville with arc lights for street lighting and incandescent lights for commercial purposes, as well as direct current for street railway operation.

The building is of brick, with a solid concrete floor and a tin roof. Current is received at 6 600 volts from the East Liverpool power-house, over two 3-phase transmission lines each 4¾ miles long.

The present sub-station apparatus is designed to take care of the commercial and railway loads, and consists of two 300-kw. rotary converters furnishing current at 585 volts for railway purposes, the current being delivered to the rotaries from the transmission line through six 110-kw. step-down transformers; and four 75-kw. transformers which deliver 2-phase current at 2 200 volts for the incandescent-lighting circuit. The arc lights are taken care of by two constant-current transformers supplied by the 2 200-volt bus. The station also contains a 200-kw. "balancer set," furnishing direct current at 500 volts for a special commercial circuit, supplying shop motors which could not be operated at the railway voltage. The station has a 9-panel switch-board.

Industry Sub-Station.—This sub-station is a new brick structure, with a concrete floor, and a reinforced concrete roof supported by

I-beams. It contains two 300-kw. rotary converters which receive current from the transmission line through six 110-kw. step-down transformers and deliver direct current at 585 volts for railway service only. Current is transmitted to this station at 13 200 volts from the East Liverpool station over a single 3-phase transmission line about 10 miles long. A 5-panel switch-board is installed.

Overhead Construction.—The usual type of span-wire construction used by direct-current interurban roads was adopted. The poles are of chestnut, from 30 to 35 ft. long, with 7-in. tops. They are set 6 ft. in the ground, with head and breast boards, and are given a deflection away from the tracks of 18 in. in 24 ft. The poles are approximately 100 ft. apart, with a minimum clearance of 24 ft. from the center line of double track, measured at the track level. The poles on one side of the road are 5 ft. higher than on the other, and carry, in addition to the trolley, the feeder and transmission lines. The butts of all poles carrying high-tension wires are charred for a distance of about 18 in. above and below the ground line. The tops and gains of all poles were painted with two coats of heavy black asphaltum before erection and before the attachment of the cross-arms. The cross-arms are painted yellow pine, fitted with locust pins, secured to the poles by through galvanized-iron bolts, and braced with galvanized-steel braces.

The trolley lines are mainly of 00 hard-drawn copper wire supported by $\frac{3}{8}$-in. galvanized stranded span wire to which they are attached by round-top hanger suspensions with 15-in. bronze soldered ears, except on curves, where the ears are carried by gooseneck suspensions with wood strain insulators. They have a clear height above the track of 19 ft. 6 in. outside of city limits and on private right of way, while in the cities this clearance is increased to 21 ft. 6 in. Protection from lightning is furnished by arresters placed on the poles at intervals of about 1 000 ft.

Transmission and Feeder Lines.—The six wires forming the double 3-phase transmission lines between East Liverpool and Wellsville are supported on one 8-ft. cross-arm, the wires of each circuit being 16 in. apart. The double transmission lines between Steubenville and Toronto are carried in delta on two cross-arms, with the apex of the triangle at the bottom, the cross-arms being spaced so that the wires form a 24-in. equilateral triangle. Between East Liverpool and In-

dustry, the single 3-phase transmission line is carried on a 7-ft. cross-arm, the wires being spaced 24 in. apart. This construction makes possible, with the least alteration, future provision for a double 3-phase line, similar to the Steubenville and Toronto lines, by simply adding another cross-arm with two insulators.

The wires are carried on brown porcelain, high-tension insulators, $5\frac{5}{8}$ in. in diameter and $4\frac{1}{2}$ in. in height, and are protected from lightning by suitable lightning arresters and choke-coils in each of the power-houses and sub-stations. The lines consist of No. 2 B. & S. hard-drawn copper wire, except between East Liverpool and Wellsville, where No. 4 wire is used. This wire is bare, except through built-up districts, where it is protected by water-proof insulation. At two points on the route the transmission lines are carried on independent pole lines which have been erected for short distances on a private right of way.

In addition to the direct-current railway feeders required for local service, the main line of the road is taken care of by 795 000-cir. mil. aluminum feeders carried on a 2-pin cross-arm below the transmission line, taps to the trolley being provided at intervals of 1 000 ft.

Cars.—New rolling stock has been purchased for the through service between Steubenville and Rochester, and consists of 18 double-truck Pullmans, 44 ft. long, finished in mahogany, with high-backed, red plush seats, having a seating capacity of forty-four. Cars of this type, and for service of this character, are usually fitted for greater carrying capacity, but in this equipment four seats, two on either side, were omitted, the seats being respaced to give more liberal distance between them, following more nearly, in this particular, steam railroad practice.

Each car is equipped with four 60-h.p. motors, mounted on Brill trucks with steel wheels. A smoking compartment, with a seating capacity of sixteen, is provided at one end; all cars are heated and lighted by electricity, and fitted with electric arc headlights, Lecturn signal lights, electric bells, parcel racks, and other conveniences.

The exterior finish is a rich yellow, lettered in aluminum outlined in black. On the letter-board over the windows, the cars bear the words "Ohio Valley Scenic Route."

Organization in the Field for Roadway Construction.—The engineering of those portions of the roadway of the East Liverpool Traction and Light Company's property constructed and reconstructed during 1905 and 1906 was looked after by Westinghouse, Church, Kerr and

Company. The work was executed under various contracts at unit prices by others and by the company's own forces.

The engineering of the subsequent roadway extensions of this company's property, the construction and reconstruction on the Steubenville and East Liverpool Railway and Lighting Company's property, and the work on the Ohio River Passenger Railway Company's property constructed during 1907 and 1908 was looked after and constructed directly by the forces of Westinghouse, Church, Kerr and Company, as was also the power work during 1907 and 1908.

The roadway organization in the field consisted of the engineer in charge, assisted by field parties and superintendents on the different main divisions, with sub-divisions under general foremen, walking bosses, foremen of gangs, etc., these being revised from time to time as the work demanded in its several parts, such as grading, masonry, track-laying, paving, and overhead work. Some unavoidable difficulties in obtaining right of way delayed the work so that the time covered was more than would have been necessary otherwise. The maximum number of men employed at one time was about 1 000. No steam shovels were used or required. Four locomotives, with about 30 dump cars, were utilized more or less continuously in the grading and in the distribution of track material and ballast. There were various concrete mixers, derricks, and pile drivers. The total cost of the roadway work for 1907 and 1908 was about $1 600 000.

The power work done by Westinghouse, Church, Kerr and Company during 1907 and 1908 aggregated about $450 000.

Government Work.—Extensive river improvements are being made by the United States Government in the construction of a deep-water channel between Pittsburg and Cairo and the building of dams and locks which will make navigation possible on the Ohio throughout the year. It is estimated that at the present time, with these improvements still far from completion, more than 10 000 000 tons of freight per annum are moved in boats and barges during the high-water stages, over the 40 miles of river between Beaver and Steubenville.

Population.—Table 1 gives the approximate population of the various cities, towns, and districts through which the road passes, or which, by reason of connections made, may fairly be considered as contributory; also the approximate mileage.

Industries.—The section of the Ohio River between Beaver and

Steubenville is rich in resources—coal, natural gas, oil, clay, quarries, etc.—which make it a natural industrial center. Here are situated some of the large iron, steel, and tin-plate works of the United States Steel Corporation and a number of independent companies; extensive plants for the manufacture of vitrified sewer pipe, fire brick, paving brick, tile and other fire-proofing materials, as well as glass-works and paper mills.

TABLE 1.

District.	Mileage.	Population.
Pittsburg	8	700 000
Intermediate	17	30 000
Beaver	4	46 000
Intermediate	11	8 200
East Liverpool	11	44 500
Intermediate	10	18 900
Steubenville	8	41 500
Intermediate	19	11 500
Wheeling	14	100 000
	102	1 000 600

East Liverpool is the principal seat of the pottery industry in the United States. Three of the twenty-seven potteries located here are said to be the largest in the country; and this industry, which alone employs 12 000 hands, with a weekly payroll of more than $100 000, produces more semi-porcelain tableware, door knobs, and porcelain electric fixtures, than any other city in the world.

Other important manufacturing interests, related or semi-related to the foregoing, make this 40 miles of river valley "a veritable hive of industry."

Amusement Park.—One of the most beautiful and popular amusement resorts in this section of the country is Rock Spring Park, at Chester, W. Va., just across the river from East Liverpool, and reached by one of the company's branch lines. It constitutes the main pleasure resort in this part of the valley, and no expense has been spared to maintain the high standard of its attractions. It is owned by the East Liverpool Traction and Light Company, and is operated by an amusement company to which it is leased.

Scenic Attraction.—The road passes through an exceptionally picturesque country, and gives promise of excellent possibilities for the development of a large excursion business; it will undoubtedly prove an attraction to pleasure seekers.

For its entire length, the route follows the river, and is located

generally on the table lands affording many fine views of the O-hi-yu or "Beautiful River," which fully justifies the name given to it by the Indian tribes formerly inhabiting its banks.

In the vicinity of Toronto, great apple orchards cover the hills for almost 10 miles, and in the spring the great mass of white and pink bloom against the fresh green of the hillside presents a strikingly beautiful picture. The beginning of the apple industry is traced to the eccentric woodsman, John Chapman, familiarly known as "Johnny Appleseed," who, during the latter part of the eighteenth century, devoted his life to planting apple trees in this vicinity.

Operation.—The various sections of the road were placed in operation as fast as completed, as follows:

Double track between Wellsville and East Liverpool, Sept. 15th,1906.

Between Calumet and Port Homer, Dec. 14th, 1907.

*Between Steubenville and East Liverpool, Feb. 11th, 1908.

Between East Liverpool and Smith's Ferry, Feb. 29th, 1908.

Between Smith's Ferry and Midland, March 21st, 1908.

Between Steubenville and Calumet, June 30th, 1908.

The entire road between Steubenville and Vanport, July 1st, 1908.

Cars are run on 30-min. schedule on the main through line, with extras and specials as required.

Financiers, Engineers, and Constructors.—The construction of these railway lines has been financed by the Ohio Valley Finance Company, the Hon. W. Caryl Ely, of Buffalo, N. Y., President. The general supervision of the whole project, the securing of right of way, and the purchase of rolling stock was looked after by Mr. Edward McDonnell, of East Liverpool, Ohio, Assistant Treasurer, and Mr. Van Horn Ely, of Steubenville, Ohio, Vice-President of that company.

Westinghouse, Church, Kerr and Company, of New York City, were the engineers of construction and reconstruction of the old properties, and the engineers and constructors of the new properties, the roadway construction being under the supervision and direction of the writer assisted by Edwin J. Beugler, M. Am. Soc. C. E. The execution of the work in the field was under the direction of William V. Polleys, M. Am. Soc. C. E., Resident Engineer. Mr. George B. Preston was the engineer in charge of the mechanical and electrical work in the field and office.

*A part of the distance between Steubenville and Calumet was operated as a single-track road until June 30th, 1908.

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PAPERS AND DISCUSSIONS

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CATENARY TROLLEY CONSTRUCTION.

Discussion.*

By Messrs. Charles Rufus Harte, W. K. Archbold, Theodore Varney, George N. Cole, W. S. Murray, and R. D. Coombs.

Mr. Harte. Charles Rufus Harte, M. Am. Soc. C. E.—The successful operation of a number of heavy-service electrified lines, together with returning financial confidence, indicate that further development in the direction of such electrification is almost certain, and make this paper most timely, particularly as there yet remains to be spoken the last word on the distribution and collection of electric energy in large quantity and at high speed.

The author's record of electrification of steam roads will apparently bear slight correction: In view of the pioneer work of Colonel N. T. Heft, it would seem desirable, in recording the New York, New Haven and Hartford Railroad electrification, to note the third-rail installation between Hartford and New Britain. The electrification of the West Shore Division of the New York Central (unless the speaker is greatly mistaken) is 600 volts, third-rail, and not overhead trolley.

The overhead construction on an electrified line is one of the very important features, for a failure on its part is most unfortunately apparent. Delays on a steam road, affecting as a rule but two or three trains, are passed over by the public as inevitable, but a failure of the overhead of the same line when electrified, with the consequent paralysis of traffic in the affected section, calls down the severest criticism, although the aggregate monthly train minutes of delay in

* Continued from October, 1908, *Proceedings*.

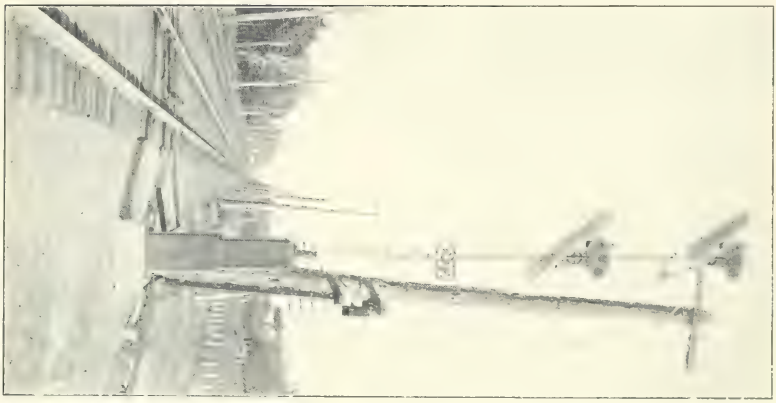


FIG. 1.—THE RELATION BETWEEN SIGNALS AND POLES.

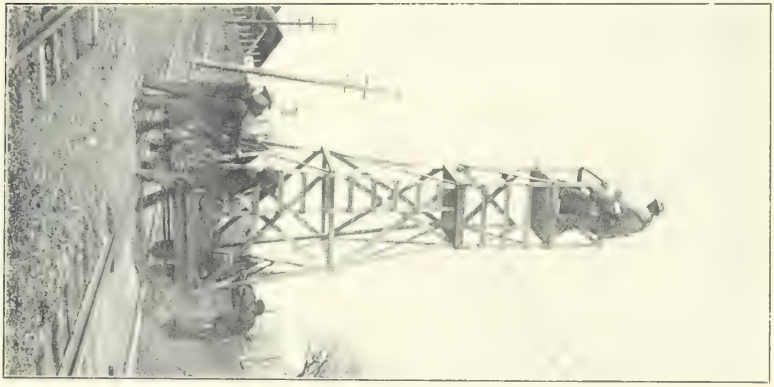


FIG. 2.—LIGHT TOWER ON GRAMPUS.

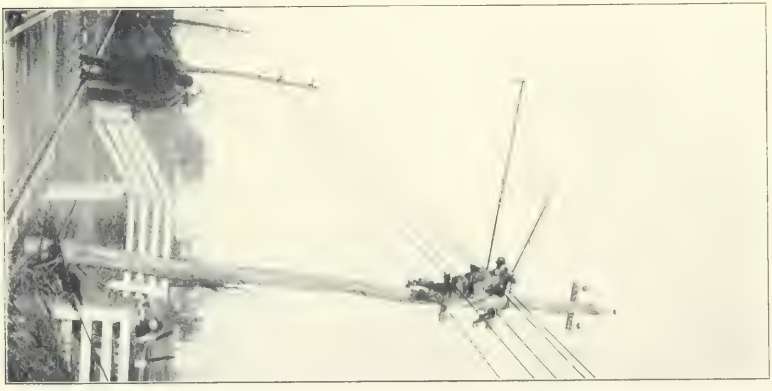


FIG. 3.—BRACKET HOIST.

the latter case are almost invariably far less than for the corresponding period under steam operation. Mr. Harte.

Not only in its general relation to the line is the overhead a critical feature. As Mr. Lyford clearly points out, the fact that it is not easily reached for inspection necessitates a class of work that will stand up satisfactorily under neglect; and this is particularly true when, as is apparently the case on the Denver and Interurban, a large portion of the line is fed directly through the trolley. The overhead, therefore, should be of the best, as regards both material and labor, and should be of such a design and character that failures are reduced to a minimum and the necessary repairs can be made with the least possible interference with operation.

This relation to operation, too, will usually be found the controlling factor in installation. Where the work train can have a reasonably clear field, a derrick car is decidedly economical for pole setting, but where there are many train movements, particularly if these are largely freights, not on regular schedule, the cost of delays on sidings will frequently be found to more than offset the mechanical gain. The lay-over loss can be reduced by using the derrick merely to pick up and place in the holes the sticks, the final setting being made with pikes while the derrick is "bottled up," but the danger of an untamped pole toppling toward and fouling the track makes the economy questionable.

The clearance of 8 ft. 6 in. from track center to pole face seems scanty. Present-day steam road practice favors at least 10 ft. as clearance distance, and this is essential to good sighting of signals, a very common location for the latter being with mast 8 ft. from rail, in which case the spectacle is in line with a pole set 8 ft. 6 in. from the center line. Moreover, with a pole spacing of 150 ft., and the poles at all near the track, there is, at high-speed operation, the optical effect of a narrow lane to which the vision of the engineer is restricted. The plan of pivoting the signal blades at the center instead of at the end assists the view materially, and has been followed on several electrifications. Aside from the question of signals, the tendency of track gangs to shift the track bodily in lining up, particularly at curves, makes it very desirable to have a little more leeway than is given by the clearance on the Denver and Interurban.

For the installation of the catenary itself some form of work train is of course a necessity, and it is desirable that this be at least the length of the standard span. The entire span can then be installed at one setting of the train, and on close multiple-point suspension there is given no opportunity for the aggravating interchange of hangers of slight difference in length. On the New York Division electrification of the New York, New Haven and Hartford Railroad, flat cars were used, a series of towers being built upon them; here the hangers are

Mr. Harte. 10 ft. apart. On the Midland Division electrification, where the hangers are 50 ft. apart, single towers were largely used, an overhang giving access to brackets. On the Berlin-Middletown-Meriden-Westfield simple suspension, the contractors, Messrs. Latey and Slater, used a regular tower car where there was little traffic, but between Middletown and Berlin, where shuttle trains kept the line hot, there was successfully used a light tower on a grampus which could be taken bodily off the track to pass trains. Brackets were erected by a simple device consisting of a pipe arm carrying the hoisting tackle and having a jaw to fit the pole, together with a galvanized-strand guy which slipped over the top of the pole. One man could readily place this, after which, by the attached tackle, ground men hoisted the heavy bracket arm and held it while it was being installed. On this line the feeders were run out by the derrick car, a sheave at the end of the boom delivering the heavy cable on the cross-arm very successfully.

As to what can be done with such methods, not much can be said that is of any value in another case. Under favorable conditions, an astounding amount of work can be done in one day, but the conditions may be counted upon to be varying somewhat in degree, and are always unfavorable.

In using a trolley wire of greater abrasive strength than copper, the author takes a step in the right direction. Joseph Mayer, M. Am. Soc. C. E., some time ago suggested the use of a mild-steel contact rod,* to be fed from the copper distribution system, and the compound catenary on the New York Division of the New Haven follows this general plan, the intermediate messenger of copper feeding a trolley of high resistance to abrasion and low conductivity. In the latter installation, by hanging the contactor from points midway between the primary hangers, using clips which grip the contactor but slide on the messenger, there has been obtained the elasticity of the Siemens-Schuckert compound catenary with a much simpler design. The results, apparently, are extremely satisfactory.

It is interesting to note the high tension used in the Denver trolley, and, particularly, the very high initial stress, which closely approximates the breaking strength of 0000 copper trolley in the brazes (6 000 lb.). The speaker specifies that 0000 trolley shall be pulled to a sag of 3 in. for 100-ft. spans at 60° fahr., and on several occasions too-enthusiastic line foremen have parted the trolley at a braze. This weakness at the braze, however, acts as a safeguard to the body of the trolley, parting before the elastic limit of the latter is reached, and as the brazes in ordinary 0000 wire are not more than 325 ft. apart, the cumulative resistances of the supports have little opportunity to be large between brazes.

* *Transactions*, Am. Soc. C. E., Vol. LVII, p. 484.

FIG. 1.—SIEMENS-HALSKE COMPOUND CATENARY. ROTTERDAM-HAAG SCHEVENINGEN LINE.

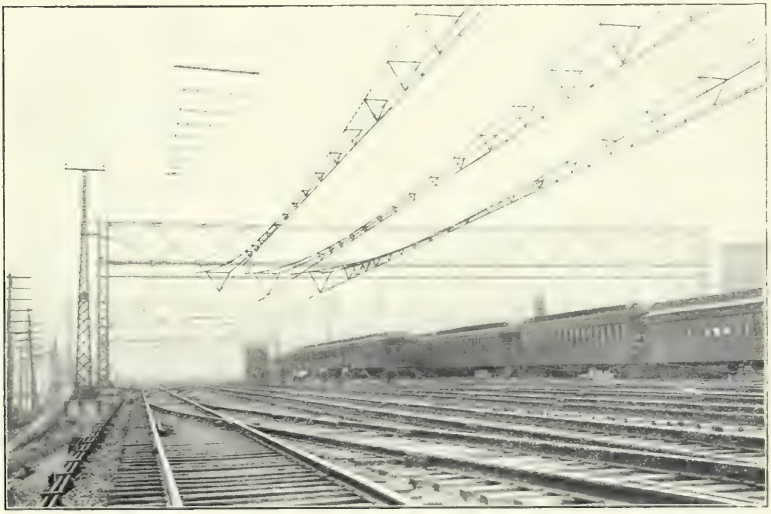


FIG. 2.—COMPOUND DOUBLE CATENARY, N. Y., N. H. & H. R. R., NEW YORK DIVISION.

Mr. Harte.

Theoretically, the high pulling stresses used in temperate weather should result in extremely severe stresses in winter, but Mr. Lyford's experience has been that of others who have pulled trolley flat in warm weather. As long as the tangents are not extremely long or the trolley anchors are not too rigid, the contraction stresses are very largely absorbed by the yield of the poles.

The splicing sleeve and special dead end (Fig. 3) are doubtless excellent, but the question naturally arises: Why not one of the market types of soldering sleeves for the first, and an eye turned in the trolley itself for the second, rather than special material?

It is the hanger, however, which really opens up the critical question: uniform characteristic of line. The trolley is a track as well as an electric conductor, and as soon as car speeds exceed 30 miles per hour the former function becomes by far the more important. Unquestionably, the absolutely rigid conductor, if smooth, would give ideal conditions, but, up to the present time, the successful development has been in the direction of the flexible type.

It is a question whether the hanger shown by the author will not by its weight cause hard spots and consequent pantagraph troubles. There is a critical speed for any type of pantagraph and hanger spacing at which chattering of the former is harmonic with the time value of the latter. Under such conditions, the blows of the pantagraph increase in violence until they lose step with the impulses, when they die down, leaving the trolley kinked at each hanger. After a number of repetitions the trolley breaks. Incidentally, on alternating-current systems, the rapid make-and-break contact tends to set up surges of tremendous energy.

To obviate the hardness at the hanger, there have been devised flexible-strand hangers, hangers sliding on the messenger, clips sliding on the hanger stem, dash-pots, and spring connections between hangers and clips. The most satisfactory kind known to the speaker is the compound type devised originally by the Siemens-Halske Company, of Berlin. As built by this company, a main messenger carries from flexible-stranded wire hangers a secondary messenger to which the trolley wire is hung by short and frequent loops which allow the trolley to rise without lifting the entire system, and at the same time permit the operation of the compensators which maintain constant tension. The New Haven's modification has screw clips rigidly connected with the trolley and free to slide on the intermediate messenger, and relying on the yield of the intermediate to give the desired flexibility, the clip coming midway between hangers, but not rising on the intermediate messenger.

The speaker would like to know what, if any, troubles have occurred from smoke deposits. Insulators close to railroad tracks condense the locomotive exhaust, which is always more or less oily; the

Mr. Harte. wet surfaces gather carbon and other dust from the smoke, and with the carbonizing of the oil because of current leakage there is formed a coat unaffected by wind or water, which lessens the capacity of the insulator very materially. Dr. Perrine, in 1904,* noted that transmission lines were largely abandoning railway rights of way as locations, for this reason; and the effect upon catenary insulation, so much closer to the source of trouble, should be, and undoubtedly is, correspondingly worse. The speaker knows of one installation where a shield has been placed beneath the insulator, but it is rather too soon to determine with what success.

The steady brace shown is excellent of its kind, but the speaker prefers a pull-off having a flexible strand. Such a pull-off can be attached to the down-curved end of the bracket if the curve is of large radius, but is preferably fastened to a bridle strung between the poles, which latter must then be on the outside of the curve. It is essential, however, that the hanger rod be sufficiently strong to stand the strain. Certain early types of hanger were deficient in this respect, and on sharp curves bent badly. On light curves it is undesirable to use small sizes of wire, but the strand used for sharper curves is so heavy that there is unpleasant sagging. Mr. Gillette, of the Washington, Baltimore and Annapolis, has cleverly overcome this difficulty by splitting the $\frac{1}{4}$-in. seven-strand cable for 4 or 5 ft., serving the main wire with one of the strands and leading one-half of the remaining strands to the top of the hanger and the other half to the bottom.

The speaker would like to know why the cross-span was put in in catenary instead of simple suspension.

The close-pole spacing on curves, to secure minimum deflection, is much to be commended, and the point that the track is tied to place by the poles is well taken, particularly with such scant clearance as in this case. Not only does a large deflection throw the wire well away from the center of the pantagraph, but the inclination of the latter, due to the curve elevation, gives a powerful horizontal thrust which tends to push the trolley still further away, with likelihood of actual movement off the pan.

At present the speaker knows of no entirely satisfactory substitute for wooden poles, but developments in steel bridges and steel poles, in connection with long spans, promise much. One of the most interesting of the pole types is being investigated and developed, but is not quite ready to be placed on the market. Of the types actually in service, the Archbold-Brady bridges, of the Syracuse, Lake Shore and Northern, have proved very satisfactory. These bridges, of channel and angle sections, carry 300-ft. spans of heavy single catenary for double tracks. It has been stated that the cost of bridges, including concrete foundations, was only $800 more per mile than standard wooden pole con-

*International Electrical Congress, St. Louis.

FIG. 1.—ANGLE-IRON BRIDGE,
SYRACUSE, LAKE SHORE AND NORTHERN RAILROAD.

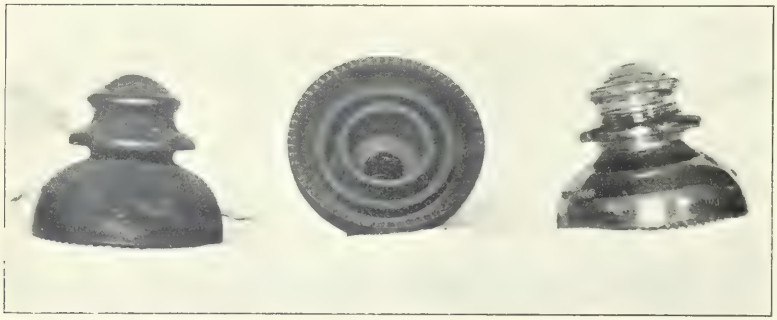


FIG. 2.—SMOKE COATINGS ON INSULATORS FROM RAILROAD RIGHT OF WAY.
INSULATOR AT RIGHT UNCOATED.

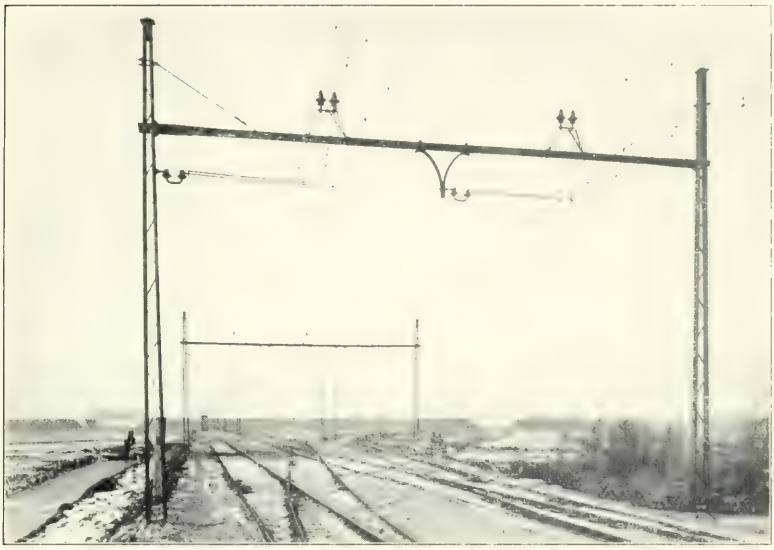


FIG. 1.—SIEMENS-HALSKE LIGHT STEEL BRIDGE, ROTTERDAM-HAAG-SCHEVENINGEN LINE.

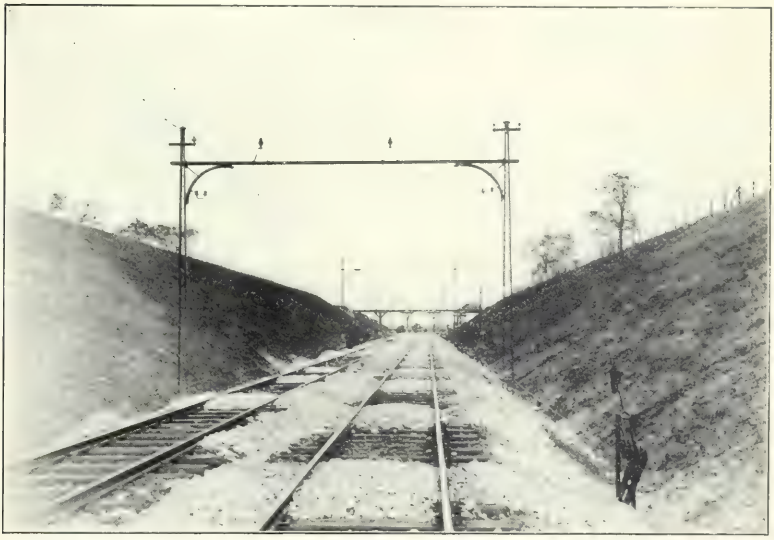


FIG. 2.—SIEMENS-HALSKE LIGHT STEEL BRIDGE,
BLANKENESE-ALTONA-HAMBURG-OHLSDORF LINE.

struction. On the same line, frames made from angles back to back were used on a viaduct. Although these were satisfactory after the catenary was installed, the structure was very flexible before it was tied to the messengers, and it is said that there was considerable difficulty in getting men to work on them. Mr. Hart.

Abroad, the Semenza transmission lines are carried on poles of angles, back to back, while the Siemens-Schuckert catenary installations have had very interesting developments in the way of bridges and bracket poles of light section. The Lancaster Branch of the Midland Railroad, of England, also has a bridge which is of the same general type. It would seem, therefore, that further development might bring out a design of low enough cost to compete successfully with wood, particularly in view of the high maintenance cost of the latter.

While it will be generally agreed that 24 ft. head-room under bridges is most desirable for electrification work, the extra 2 ft. will hardly be secured in New England without heavy expense, even in the open country, while, in the more thickly settled portions, property damage offers a most serious bar. By setting 24 ft., however, for the standard, and working as nearly to this as possible, one of the serious overhead problems will be materially simplified.

W. K. Archbold, Esq.—The speaker has been much interested in this paper, as he has had to meet problems of the same class. He agrees with the author that a reasonable-priced substitute for wooden poles for catenary work is desirable, and believes that steel construction can be provided at a sufficiently low cost. Catenary and transmission-line work require poles of large sizes and of good quality, and it is difficult to procure such poles in large quantities. Mr. Archbold.

The cost of renewals of wooden poles is likely to be a serious element in maintenance in the near future. The speaker is familiar with two roads, not yet four years old, where sawed-pine poles were used, and about half the poles have either been renewed or will require renewal in the near future. The cost of such renewals, especially where high-tension lines are also carried, is of course high and the work dangerous. Timber is becoming scarce, and, while chestnut poles seem to have longer life in the North, they are usually crooked and are also growing scarce.

In discussing Mr. Coombs' paper,* the speaker described some single catenary construction on bridges 300 ft. apart, on the Syracuse, Lake Shore and Northern Railroad near Syracuse, N. Y. While it is now decided that the road is to be operated permanently by direct current with wheel trolleys, instead of 6 600-volt, single-phase current, as first planned, the bridges and catenary construction are to be used for a 14-mile extension to Phœnix and Fulton. The bents on the

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 547.

Mr. Archbold. bridges have been lightened somewhat, the trusses remaining practically as before described.

For a comparison of cost, the speaker would call attention to Fig. 1, Plate CXLI, showing standard wooden-pole construction on double-track road, as specified by T. H. Mather, M. Am. Soc. C. E., under whose direction the catenary construction has also been erected. The wooden construction is cross-span, using not less than 35 ft. 8-in. top poles, with a standard spacing of 85 ft. on tangents.

Fig. 2, Plate CXLI, shows a catenary bridge construction. Based on item prices and records, the additional cost of the bridges and catenary trolley over the actual cost of the standard wooden construction shown, with ordinary trolley, is $800 per mile of double-track road.

In addition to the statement made by the speaker in discussing Mr. Coombs' paper, it can still be said that, thus far, there has been absolutely no difficulty from side sway on the 300-ft. spans. It was stated at that time that there was yet doubt as to whether or not the line would prove to be too stiff in the vertical plane. There was some difficulty in the hot weather, and it was observed that a part of the line on which the hangers were spaced at 20-ft. centers as an experiment worked better than where they were placed at 10-ft. centers. It was determined, therefore, to go one step further, and that the new construction should have the hangers placed at 30-ft. centers, and that enough hangers should be taken off the first section to give them the same spacing. Experience proves that this is much better. Standing on the rear platform of the car, and holding the trolley rope, a considerable chattering could be felt when the spacing was 10 ft., and, with a heavy car, considerable arcing could be observed at the trolley wheel. With the 30-ft. spacing, a slight rise and fall of the trolley wheel can be noticed by holding the rope, but there is no chattering effect.

The Ohio Brass Company's insulators, sister-hooks, rods, and Detroit clamps used on the first construction will be used on the extension. There was a question as to whether the use of the porcelain insulators should be continued, but as there has been no difficulty from the breakage of these insulators, and as the composite trolley insulating materials deteriorate in time, Mr. Mather decided to make no change in the type of insulators, although, at the maximum, the voltage will be 600, direct-current.

In the operation of work trains on steam railroads, the speaker's company has found the same difficulty as described by the author. Frequently, only from 3 to 5 hours' work a day can be obtained from a work train with the linemen assigned to it. The charges for construction trains run from $25 to $50 per day, and the payroll of the construction force may easily be as much more. Experience shows that as much work as possible should be done independently of the

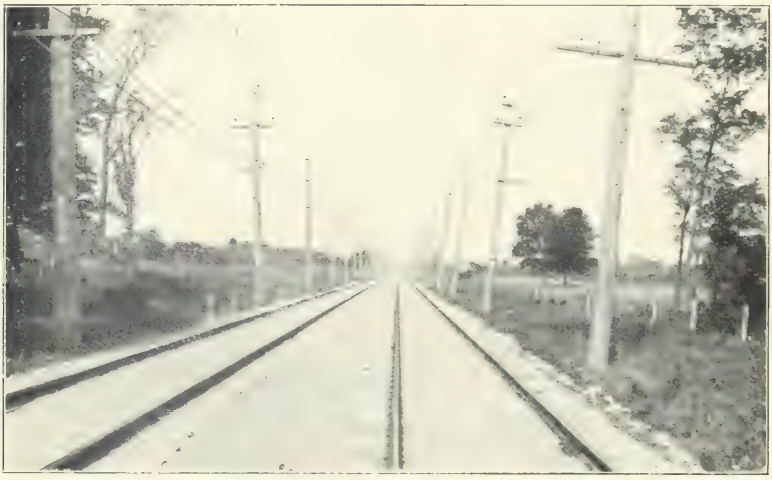


FIG. 1.—STANDARD WOODEN POLE CONSTRUCTION ON DOUBLE-TRACK ROAD.

FIG. 2.—CATENARY BRIDGE CONSTRUCTION.

train. It will be necessary, of course, to deliver bridges, or poles, and to string out the wires from trains during the day time. Where traffic is heavy on a road, it would perhaps be economical to set the hangers at night, and better and more economical results can be obtained, after the wires are strung, by pulling them up with ropes and blocks, with a team or by hand. Mr. Archbold.

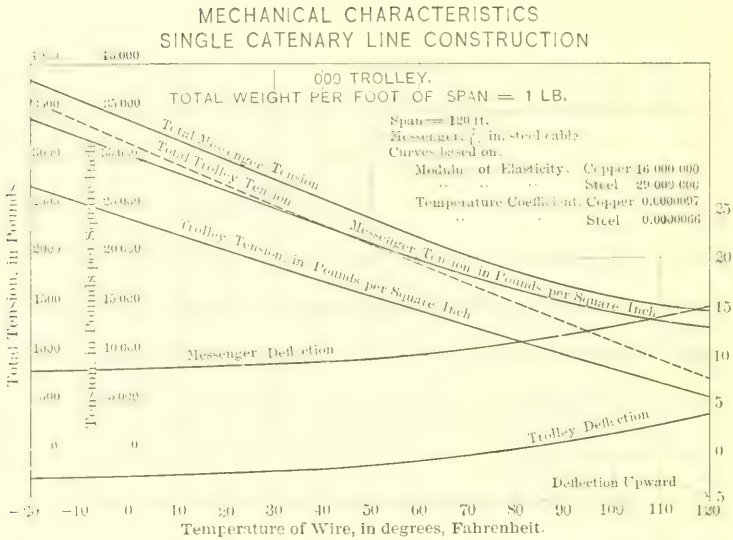
The use of a dynamometer in stringing wire has not seemed practicable, in the speaker's experience, and the best results have been obtained by sighting the wires and using a sag table with proper corrections for temperature changes. The large difference in initial strains in pulling the messenger cable, mentioned by the author, are not so important in the work on the bridges with 300-ft. spans, as there are, of course, fewer points of support to make friction on the wire, and the bridges are more stable than bracketed poles. In the work mentioned, use was made of pulleys, which were attached to the top chords of the bridges, and over which the messenger cables could be run. Even with this plan, there would be some difference in initial strains, but, of course, it would be reduced to a minimum; and, to permit the spans to equalize, the messenger wire should not be tied in until the expiration of a day, or, preferably, two or three days, after it is strung.

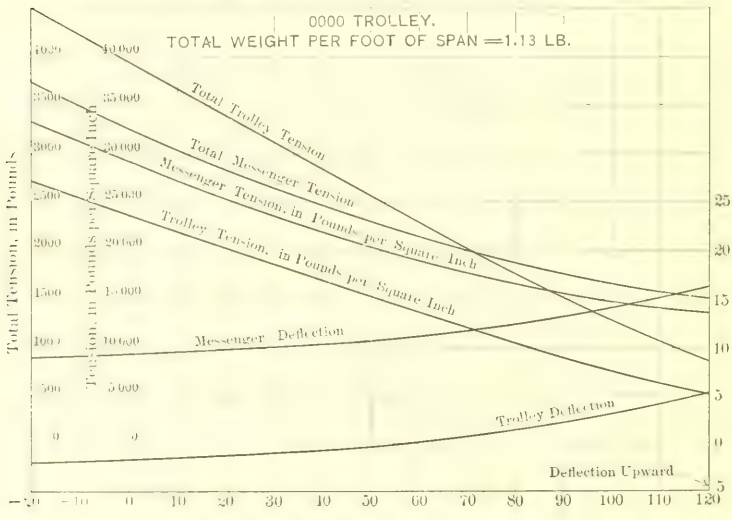
THEODORE VARNEY, ESQ. (by letter).—Referring to Mr. Lyford's statements regarding the rapidity with which the construction work was carried out on the Denver and Interurban Railroad, his final rough estimate—$3 500 to $5 000 per mile of single-track construction—appears to be high. This, however, as he says, is probably due largely to interruptions and other contingencies affecting the labor item. It would be interesting to know what the author's estimate would be under the same conditions, providing no regular service had to be maintained during construction, as, for instance, on a new road. Mr. Varney.

Referring to the question of sag in the messenger cable and trolley wire, the author is undoubtedly correct in his idea of having the trolley wire and messenger cable taut at all temperatures, but it would seem that 2 000 lb. at 100° is tighter than necessary. The diagrams, Figs. 7 to 10, indicate conditions which appear to be entirely practicable. In these curves the tensions in the messenger cable correspond to the load produced by trolley and hangers. In Europe, where automatic tension devices are used, it may be noted that a tension of about 1 100 lb. is maintained on a wire a little less than 000 in size.

It is believed that the pole spacings are rather closer than are actually required to keep the wire on the pantagraph contact. In the case of a new road, it is undoubtedly best to limit the "stagger" at first until the roadbed has settled, but afterward more deviation than that given in Table 1 is believed to be safe, and will result in longer life to the shoes. It would be interesting to know the length of life

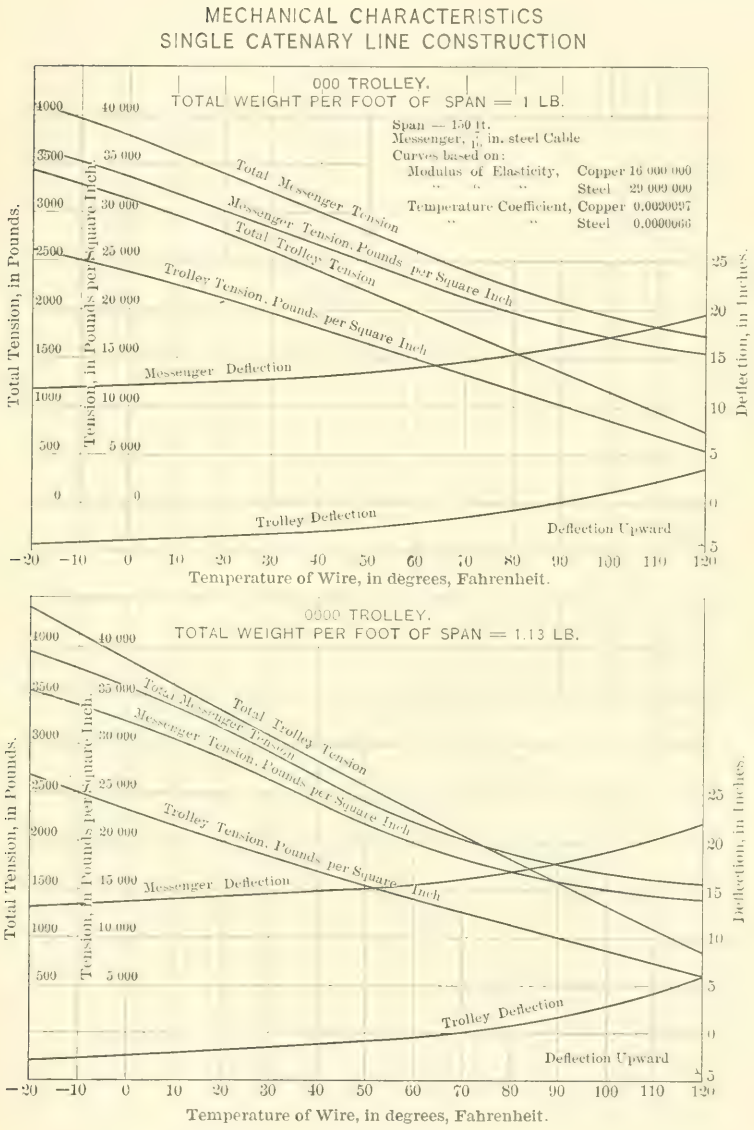
Mr. Varney


MECHANICAL CHARACTERISTICS
SINGLE CATENARY LINE CONSTRUCTION
000 TROLLEY.
TOTAL WEIGHT PER FOOT OF SPAN = 1 LB.
Span = 120 ft.
Messenger, $\frac{7}{16}$ in. steel cable
Curves based on:
Modulus of Elasticity, Copper 16 000 000
" " " Steel 29 000 000
Temperature Coefficient, Copper 0.0000097
" " Steel 0.0000066
Total Messenger Tension
Total Trolley Tension
Messenger Tension in Pounds per Square Inch
Trolley Tension in Pounds per Square Inch
Messenger Deflection
Trolley Deflection
Deflection Upward
Total Tension, in Pounds
Tension, in Pounds per Square Inch
Deflection, in Inches.
Temperature of Wire, in degrees, Fahrenheit.


0000 TROLLEY.
TOTAL WEIGHT PER FOOT OF SPAN = 1.13 LB.
Total Trolley Tension
Total Messenger Tension
Messenger Tension, in Pounds per Square Inch
Trolley Tension, in Pounds per Square Inch
Messenger Deflection
Trolley Deflection
Deflection Upward
Total Tension, in Pounds
Tension, in Pounds per Square Inch
Deflection, in Inches.
Temperature of Wire, in degrees, Fahrenheit.

FIGS. 7 AND 8.

Mr. Varney.


MECHANICAL CHARACTERISTICS
SINGLE CATENARY LINE CONSTRUCTION
000 TROLLEY.
TOTAL WEIGHT PER FOOT OF SPAN = 1 LB.
Span — 150 ft.
Messenger, $\frac{1}{4}$ in. steel Cable
Curves based on:
Modulus of Elasticity, Copper 16 000 000
" " " Steel 29 000 000
Temperature Coefficient, Copper 0.0000097
" " " Steel 0.0000066
Total Messenger Tension
Messenger Tension, Pounds per Square Inch
Total Trolley Tension
Trolley Tension, Pounds per Square Inch
Messenger Deflection
Trolley Deflection
Deflection Upward
Total Tension, in Pounds.
Tension, in Pounds per Square Inch.
Deflection, in Inches.
Temperature of Wire, in degrees, Fahrenheit.
0000 TROLLEY.
TOTAL WEIGHT PER FOOT OF SPAN = 1.13 LB.
Total Messenger Tension
Total Trolley Tension
Messenger Tension, Pounds per Square Inch
Trolley Tension, Pounds per Square Inch
Messenger Deflection
Trolley Deflection
Deflection Upward
Total Tension, in Pounds.
Tension, in Pounds per Square Inch.
Deflection, in Inches.
Temperature of Wire, in degrees, Fahrenheit.

FIGS. 9 AND 10.

Mr. Varney. of a pantagraph shoe on the Denver and Interurban Railroad. Deviations of at least 7 or 8 in. on each side of the center should be safe for a shoe 4 ft. long on the flat portion.

The question of grounding the bracket arms is one which is deserving of careful consideration. In cases of roads with two or more tracks; where the lines may be broken into comparatively short sections; where the traffic may be diverted to other tracks, while one is tied up for repairs; and, especially, where the necessity for great stability and long spans demands steel supporting structures, a grounded support for the insulator is not objectionable. In these cases, however, automatic devices should be used in order to isolate the grounded section and not tie up the entire system when an insulator breaks. The case is different, however, for a single-track road without automatic sectionalizing devices and with little attendance. Broken insulators are now being replaced on some roads, with 6 600 volts on the line.

It is believed that it would pay to investigate methods of impregnating wood poles in order to make them weather-proof and also fairly good insulators. In every pole 20 ft. of good insulation is being wasted, and this, in case of a broken insulator, should suffice to keep the road operating until the insulator can be replaced.

Mr. Cole. GEORGE N. COLE, ASSOC. AM. SOC. C. E. (by letter).—It is very evident from this paper that one of the main objects in the care used in the erection of trolley lines is to obtain a perfectly level trolley, eliminating sag and swaying.

By eliminating sag a smooth-running pantagraph at high speeds is obtained, and, to eliminate this sag, frequent trolley hangers are necessary. Reference has been made to the fact that at certain speeds a considerable vibration is set up in the pantagraph which causes it to hammer the wire with considerable force, and Mr. Harte mentions a trolley wire which was badly bent at its support from this very cause.

It is evident that this vibration is induced when the rate of passing the trolley hangers is the same as the rate of vibration of the pantagraph and its springs, or some multiple or harmonic of it.

Since, from its present construction, the pantagraph has inevitably a rate of vibration, it occurs to the writer that vibration of the pantagraph would never be started if the trolley hangers, from the messenger wire to the trolley wire, were placed so that no regular rate of passing them would ever occur.

This can readily be accomplished by spacing these trolley hangers at such irregular intervals that they will fail to pass the pantagraph in a rhythmic manner or set up a beat in it.

Care should be taken that this variation in spacing is not made a fixed or regular variation, or a higher vibration might be set up in what one might call an overtone, and give rise to an objectionable hum, or worse.

W. S. MURRAY, ESQ.* (by letter).—This paper has naturally brought to the writer's mind the allied features common to the New Haven electrification. It is, of course, to be noted that there is a wide dissimilarity between the form of construction adopted on the New Haven system and that described by the author. Both, however, are tied to a common factor, namely, the frequency, phase, and voltage, and although, even here, certain detailed electrical operating conditions are not entirely similar, the closeness of the relation may make valuable the citation of some experiences on the New Haven system. Mr. Murray.

The writer approves heartily of the methods relative to construction in the field, so well described in this paper. Doubtless Mr. Lyford has found out that the work train is the "heart-wringer" of the operating department. It requires the highest order of intelligence to control its movements, and the labor in connection with it is paramount. It is most important that the work-train organization be of a character to insure the maximum possible number of work-minutes during the day and the least number of minutes of interference with the revenue train schedule; the roster of each man's time must be laid out by previous schedule, and not arranged on an *ex-tempo* basis in the field. In reviewing the time rates Mr. Lyford has mentioned, concerning the erection of poles and brackets, it would seem that just such a perfection of work-train schedule had been secured. The cost of labor can jump in factors of two, three and, indeed, as high as four times the minimum if every care is not exercised in the outlay of the time and the location of the work-train service. Again, emphasis should be placed on the necessity of the contractor remembering that the train despatcher can be either his best friend or his worst enemy, and, if the latter, he can easily be forgiven, as there is nothing that makes such havoc in a railroad schedule as a work train.

Mr. Lyford's summation of the several railroad companies who have trains under electrical operation or have framed definite plans for their operation by electricity is of great interest, and the fact that, out of the number mentioned, 50% have chosen single-phase electrification proves the directional tendency of the new art. Many have already grown gray-haired over the period in which car or train propulsion has been obtained by the direct current, and, indeed, all may be proud of the splendid achievement and the degree of perfection secured in this form of electric propulsion. As a matter of fact, its very limitations have been the cause of its perfection, and in the new art may be seen this high notch of direct-current perfection, the starting point for greater attainments to be secured in the application of single-phase traction.

There is no little pleasure in the realization of a prophecy the writer felt so sure would be fulfilled, namely, the approval by the Public Service Commission of the high-tension wires erected on the

* Electrical Engineer, New York, New Haven and Hartford Railroad.

Mr. Murray. right of way of the New York Central Company within the limits of the City of New York. In the recent decision of Commissioner Eustis may be found a just and careful reasoning. The greatest argument against the use of high-tension wires within city limits is prejudice. Analyze prejudice and invariably the fibers of its structure disintegrate. Officials who are above prejudice, and judge a situation absolutely on its merits, must be constrained to decisions in which the public and the railroad company's interests are fairly equated. It is no strain of conscience or discernment to realize that frequently the railroad's interest is the public's interest.

The writer fears that he is deviating slightly from a discussion of this paper, and yet Mr. Lyford says:

"The principal merit in the single-phase system lies in the ability to operate at high voltage with a single overhead conductor, and, therefore, to transmit power long distances with a minimum number of sub-stations and a minimum quantity of copper. These arguments apply particularly to trunk-line operations, and this system, therefore, appeals especially to the managers and engineers of large railroads."

Concerning the use of phono-electric trolley wire *versus* hard-drawn copper, the writer cannot quite agree with Mr. Lyford in the reasoning by which he arrives at the unimportance of its low conductivity, in virtue of the higher resistance of the steel rail to the path of the current; the writer would rather be led to the necessity of keeping out any further extra resistance. It is quite true that, by the introduction of this higher rail resistance, the percentage of drop in the overhead wire to the total circuit drop is less, but it is the total and not this percentage which is of the greater importance. However, notwithstanding this reasoning to the contrary, between the two wires, the writer is in favor of the choice made by Mr. Lyford in the use of the phono as a contact wire (conductivity being supplied by other means), for, as such, experience indicates that, though always higher in cost per pound, it is not only cheaper than copper, as its life is more than twice as long, but its reliability and resistance to fracture is also one of its marked advantages.

To return to the form of construction described by the author, its successful operation is dependent entirely on the speeds to be made. If such speeds as have obtained on the New Haven tracks are to be negotiated on Mr. Lyford's line, the writer would prefer the form of suspension the New Haven road has recently adopted. It is unquestionably more expensive that the type described by Mr. Lyford, but it fulfills one of the requirements laid down by him under "sufficiency," with which the writer is in full accord, namely:

"High-voltage catenary work cannot receive the close daily inspection of the track-walker, and, because this is so, there is the greatest necessity for thoroughness and permanence in the construction."

Mr. Murray.

There is a perfect analogy between a catenary form of overhead trolley construction and a regular highway suspension bridge. If a highway bridge of the suspension type is to be built, the engineer first asks what is to be its traffic density; how many car tracks are to be laid on it; how many wagons are to be permitted to pass; what load is to be concentrated at any point; in short, what is to be the commercial duty of the bridge? Having obtained this information, he then designs the cables and general members of the bridge, and finally the floor, the latter usually being cheap and quickly and easily replaced. The electrical engineer likewise wants to know something about the traffic on his bridge. He will certainly want to know what the current density is to be, and, with this information, he must design sufficiently strong cables to hold up his copper conductors and take care of the current traffic; and, finally, why should he not design the floor of his bridge, namely, the cheap easily replaceable iron wire? Can it not be agreed that electrical engineers have been making the mistake of running on the members instead of the floors of their bridges, and thus slowly, but surely, decreasing the (current) traffic capacity of their structures. If the engineers of the highway suspension bridges were to run on the members of their structures, the bridge would soon fail; thus, by the use of the auxiliary wire, one approaches, if he does not reach, a state of the electrical art which has for its guarantor many years of mechanical experience.

The relative life of phono *versus* steel wire is one of the most absorbingly interesting details, and later the writer hopes to be able to offer some statistics which may be valuable to those engaged in electrification. There is no more important link in the distributing system than the contact wire. Next to the traffic rails, it is the most abused piece of metal exposed, and it deserves a high order of consideration. The writer agrees most heartily and earnestly with the author in his statement that hard spots in the catenary construction should be eliminated. Indeed, they make inoperative a high-speed line, and, considering the hard spots individually, the writer would name, in the order of their degree of menace to high-speed operation, deflectors, section breaks, and hangers. Engineers have eliminated the hard spots due to the hanger, and are studying how to eliminate the others. It is a happy provision that the two latter are comparatively few as compared to the first named. The writer has every reason to believe that the same success obtained in cushioning the hard spots, due to hangers, can be secured in the proper design of steady strains, deflectors and section breaks. In any one of these classes of construction an adherence to the general form of line construction should be followed.

It is pleasing to note Mr. Lyford's choice of 4 or 5 miles as sectionalizing distances. Actual operation has proved the advantage of this distance, and, although originally sections averaged 1½ miles in length,

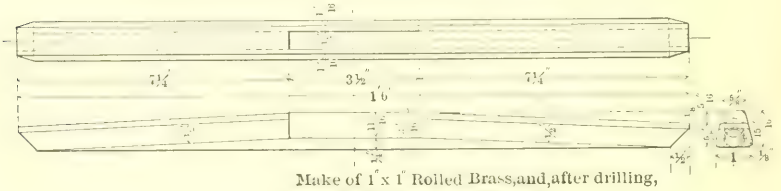
Mr. Murray. in some instances, this distance is now increased to the figures above named.

It is interesting to note the figures, mentioned by Mr. Lyford, covering single-phase electrification; the writer is able to vouch for their conservativeness. The careful record of expenditures made, to date, upon a four-track electrification has presented a distribution of costs most interesting to study. The entirely successful experience undergone in the past year with the cross-catenary form of construction in the East Portchester yard, where as many as ten tracks are spanned, strengthens the writer's previous belief that in the future this form of construction will have excellent application to roads having four or more tracks, upon which occasion he believes the cost of electrification per single track will be considerably below the figures named by Mr. Lyford.

Mr. Coombs. R. D. Coombs, M. Am. Soc. C. E.—In connection with the detail of the trolley splicing sleeve shown by Mr. Lyford, the following tests recently made during the installation of the experimental line of the Pennsylvania Terminal and Tunnel Railroad on Long Island, may be of interest.

On account of the small vertical distance between the secondary messenger and the trolley wire, a compact splice was required, and four samples were made and tested.

TROLLEY WIRE SPLICE GROOVED 0000 COPPER



Make of 1" x 1" Rolled Brass, and, after drilling, machine to dimensions shown

Fig. 11.

The first sample was identical with that shown in Fig. 3; the second was like the first, except that the bottom surface was rounded; the third and fourth were as shown in Fig. 11. It was the intention to solder short lengths of 0000 grooved trolley wire in each sleeve and then test them to destruction in a tension machine. On attempting to solder the first and second samples, the material cracked along the middle of the bottom surface for a distance of about 6 in. from the end. The third and fourth samples were soldered without difficulty, using 0000 hard-drawn grooved copper wire for the former and 0000 medium-steel grooved wire for the latter.

No. 1.—Cracked in soldering.

No. 2.—Cracked in soldering.

No. 3.—0000 copper, 7 630 lb. ultimate strength, 46 000 lb. per sq. in.

No. 4.—0000 steel, 11 630 lb. ultimate strength, 72 000 lb. per sq. in.

In both instances failure occurred in the trolley wire outside the splice. Two additional pieces of the wires used were tested and gave substantially the same results, showing that no injury was done to the wire in soldering. Mr. Coombs.

By planing off the upper corners, as shown in Fig. 11, the speaker attempted to lessen the weight of the splice and obtain if possible some slight elasticity at the ends of the sleeve. It would appear, from these tests, that the removal of this unnecessary material tended to prevent unequal contraction and cracking.

As stated by Mr. Lyford, intelligence and experience are needed in the design of overhead material, and the cost of time spent in developing workmanlike designs will be saved over and over again in the field. Not only the simplicity of each aerial erecting operation, but also the number of operations, must be considered. Assuming that a given catenary detail is mechanically able to perform its function, that it will not interfere with the desired elasticity of the line, and that it induces a minimum of aerial work, it will still be undesirable unless free from wear and corrosion.

This question of maintenance deserves more careful consideration. On lines of little traffic repairs may be possible at a reasonable cost, but it is not believed that the average operating department would view with composure the need for much work-train service on a busy line.

The life of galvanized material exposed to the elements or to locomotive gases, or to both, is a subject upon which there is great difference of opinion among engineers. If it is a fact that galvanized material cannot withstand the atmosphere of the coke regions for even a short period, there would appear to be urgent need of tests before galvanizing alone, or in conjunction with a carbon deposit from the locomotive exhaust, is accepted as sufficient protection. Further, the value of the usual tests of galvanized material is open to some suspicion, if the application of a minute quantity of grease will enable a rejected sample to pass the test.

The speaker is of the opinion that longer spans and fewer supports are desirable. As stated in a previous paper,* the multiplication of posts near the tracks is objectionable in appearance, increases the risk from derailments, interferes with the view of signals, and is subject to considerable discussion as a measure of economy.

Theoretically, as the cost of the material between the supports is relatively low, much longer spans would be economical. The principal objection to long spans is founded upon the possibility of lateral displacement by wind. This is undoubtedly possible, but, in the speaker's opinion, it has been given undue importance in various designs; in fact, he believes that even the 300-ft. spans recently installed on two roads in the United States might have been increased with advantage.

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 505.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE HYDRO-ELECTRIC DEVELOPMENT AND TRANSMISSION LINES OF THE CANADIAN NIAGARA POWER COMPANY.

Discussion.*

By Messrs. John C. Parker, and William J. Boucher.

Mr. Parker. John C. Parker, Assoc. M. Am. Soc. C. E.—This paper gives a very vivid description of one of the most finished hydro-electric plants. The Society and the profession are to be congratulated that such excellent detailed drawings, photographs, and intimate information are obtainable.

Some slight additional interest attaches to this paper when viewed in conjunction with a paper delivered by P. N. Nunn, M. Am. Soc. C. E.† Mr. Nunn's paper was descriptive of the Ontario Power Company's plant, which is quite closely contiguous to that of the Canadian Power Company. Familiarity with these two plants, and the descriptions, indicate along what diametrically different lines two developments of a similar degree of excellence may be developed. It is interesting to note how, from the fundamental character of the locations of the two plants, entirely different emphasis of detail has been impressed upon the designs. Where one plant is developed with a wheel-pit and tail-tunnel, the other is built in the open, in the river gorge, with but short tail-races and a very long supply flume. One plant uses vertical-shaft turbines and generators with elaborate oil pressure and lubricating systems, the other uses horizontal-shaft units,

* This discussion (of the paper by A. H. Van Cleve, M. Am. Soc. C. E., printed in *Proceedings* for August, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† At the Annual Convention of the American Institute of Electrical Engineers at Asheville in 1905.

both members of the unit being contained in the same room and with but a rudimentary lubricating system. One plant represents a highly concentrated development, the other a geographically extensive layout. In one case protection from ice is less nearly automatic than in the other, but, at the same time, has involved materially simpler intake works. In one case control of the operation and of the electric circuits is entirely in the power-house, the transformer station being quite subordinate; in the other the transforming station becomes the seat of control. It will be interesting, as time advances, to note the operation of these two most excellent plants with reference to the numerous points of dissimilarity in the detail. Mr. Parker.

In discussing the dimensions of the draft-tubes, the author, referring to the high discharge velocity of 21 ft. per sec., suggests the utilization of this velocity in imparting to the discharge water a part of the velocity required in the tunnel because of the angle made by the draft-tubes with the tail-race. The speaker would like to inquire whether, by gradual enlargement of the draft-tube just before it enters the tail-race, and especially by making the enlargement on the down-stream side, much of the shock would not be obviated? There are doubtless adequate reasons why this has not been done, but it seems that no great portion of the velocity head would be recovered in the impact.

The formation of the inlet and outlet casings of the turbine are of much interest. It seems rather surprising that in such large units the manufacturers should have designed a concentric casing in which flow to and from the runner must, of necessity, be clockwise in some parts of the casing and counter-clockwise in others. This must involve either additional work by the diffusion vanes and special construction of the various vanes around the periphery, or considerable shock in the inlet and discharge, and would seem to involve necessarily the localization of the flow into the runner. As contrasted with this form of casing, it is interesting to note the vortical type sometimes used, both in turbines and in the inverted form of the turbine—the centrifugal pump. Unquestionably, in view of the vast and uniformly creditable experience of the manufacturers, these details have been provided for, but it would be interesting to know what means have accomplished these ends, or whether they have been sacrificed to other and more important desiderata.

It would be interesting, also, to inquire what provisions or considerations of design have been brought to bear on the matter of acceleration and retardation in the discharge tunnel. Ordinarily, a very considerable reactive effect on the governors might be expected, when, owing to the sudden closing of the gates, the tail-race discharge, under its momentum, tends to lower the water level in the race, increasing the head and thereby offsetting in part the operation of the

Mr. Parker. governor; similarly, with a sudden increase of load, there would be a piling up of water in the tail-race. With designs of a certain character this might be expected to cause racing or to augment the duty required of the governor. The throttling gate at the lower end of the tail-race in this plant, of course, tends to minimize such difficulties on the closure of the wheel gates, which is the more severe condition, but, on the other hand, under the less severe condition of opening, would augment any such tendency. It seems quite probable that in this particular case the balance of the effect from the throttling would be favorable rather than otherwise.

The transmission structures are novel and striking in many details. The construction would seem to be a remarkably simple one, easy of erection and maintenance. One detail in particular seems to be most excellent, and that is the comparative ease with which the cross-arms can swivel on the top pipe, relieving the structure of an otherwise unsupportable strain, longitudinally of the line, in case of breakage of one cable. It is interesting to notice also that provision has apparently been made for carrying an overhead grounded cable for lightning protection. It would be interesting to know whether this has been installed, and, if so, what conclusions the power company has arrived at with reference to the value of such protection.

As to galvanized and painted structures, the speaker believes that the balance of favor is with the former, especially where two circuits are carried on one line of poles, rendering repainting without interruption of service a difficult matter. Some time ago, in preparing figures with reference to this point, the speaker found that on a certain structure, which it would cost $4 to paint galvanizing could be done for only $11. The experience of windmill manufacturers has been that hot galvanizing, properly applied, will easily last practically intact for 20 years, so that the cost of repainting during this period would probably exceed the initial cost of galvanizing.

With reference to the span, it has been found quite feasible on several transmission lines to go to 550 ft., and this span will doubtless be exceeded with the advent of higher voltages and more costly insulators, especially where the latter are made of the underhung type. On the other hand, of course, an upper limit must speedily be reached, especially with aluminum cables, because the sag of the latter, markedly in the smaller sizes, rapidly increases with the span, and the tower cost goes up even more rapidly as the height is increased. In a line recently placed by the speaker, using aluminum cable of 214 000 cir. mils, it was found necessary to use a sag of 24.5 ft. when stringing at summer temperatures to a span of 550 ft. This involved a tower 56 ft. in height and necessarily a fairly heavy construction, as the width of the right of way and of the permissible tower base was restricted.

The manhole and duct construction shown in Fig. 6 is in striking

contrast to the ordinary construction adopted by distribution companies, and perhaps would be economically impossible for such purposes. It emphasizes one fact, however, which ordinarily has received all too little consideration, and that is the impossibility of getting rid of the heat produced in the center of a large block of ducts. The speaker believes that as time progresses even the small distribution companies will learn that such grouping is not good. Mr. Parker.

Much speculation has been indulged in with reference to the swinging of transmission cables as a result of gusts of wind. Experience has proven that on large spans supported on towers the cables swing synchronously, owing probably to the forced vibration produced by the reaction of one cable on the other through the more or less elastic tower. The Niagara River crossing, however, presents a modification of the ordinary condition in that the cables are not rigidly attached to the towers, and hence are not likely to communicate to the towers the elastic stress produced by the wind. In fact, the whole philosophy of the counterbalance weight is to keep this stress—or the vertical dip of the cable—constant. It would be interesting to know whether, in this crossing, the wind strains have been felt uniformly by all the cables, so as to permit synchronous swinging and hence to eliminate the danger of lines coming into contact.

William J. Boucher, Assoc. M. Am. Soc. C. E.—The author is to be congratulated on this very interesting and valuable paper. The Canadian plant appears to resemble that on the New York side in many particulars, but it differs from the latter in so many ways that Mr. Van Cleve's description forms a very desirable addition to the literature on hydro-electric plants. Mr. Boucher.

Mention is made of the market for the electric current, and, in addition to its uses in comparatively near localities, it is now transmitted as far east as Syracuse, a distance of 150 miles, where it operates the entire street-railway system. Current is also supplied to about half of the electrified portion of the West Shore Railroad operating between Syracuse and Utica, which makes the total distance of transmission nearly 175 miles.

The "creeping" or "moving" of the walls of the wheel-pit is a very curious phenomenon and, if Mr. Van Cleve is prepared to give further information, it would be interesting to learn if a satisfactory cause has been assigned. How much have the walls moved since the completion of the excavation? Is it expected that the creeping will continue, and if so, at what rate; and did similar creeping take place at the plant on the New York side?

With reference to the method of excavating by channeling the sides, was the central core afterward blasted out? Figures of cost of excavation by this method, if the author is at liberty to give them, would be instructive.

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PAPERS AND DISCUSSIONS

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FORESTS AND RESERVOIRS IN THEIR RELATION TO STREAM FLOW WITH PARTICULAR REFERENCE TO NAVIGABLE RIVERS.

Discussion.*

BY MESSRS. F. COLLINGWOOD, THOMAS P. ROBERTS, STEPHEN CHILD, L. J. LE CONTE, F. B. MALTBY, J. FRANCIS LE BARON, AND EDWARD P. NORTH.

Mr. Collingwood.

F. COLLINGWOOD, M. AM. SOC. C. E. (by letter).—In these days of unrest, it would seem that no conclusions in any realm of thought or observation are so firmly established but that some acute thinker will bring forth arguments to prove their unsoundness.

This is the case in this interesting paper, but, while acknowledging the force of the arguments advanced, the writer is unable to accept the author's deductions as a whole.

Four propositions are given, as covering the ordinarily received opinion as to the effect of forests on stream flow.

The first of these relates to the effect of the humus acting as a sponge which absorbs the rain and melting snow and pays out the water in a more equable flow. The word, "humus," does not fully cover the point involved, as consideration must be taken also of the sponge of dead leaves, and, in many cases, the great depth of moss growing upon the surface. Then, as helping in a marked degree to resist soil erosion, the matted roots of a forest must not be overlooked.

* This discussion (of the paper by H. M. Chittenden, M. Am. Soc. C. E., printed in *Proceedings* for September, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Collingwood.

These are particularly important in the case of a thin soil cover resting on sloping rock. Observation would seem to show that the young growth does not fully take the place of large trees, particularly in such locations.

It must be admitted that in time of extensive and heavy downpours of rain, when the ground is practically saturated with moisture, the effect of forests as regulators of stream flow is at a minimum; but it is not believed that it is ever entirely without effect. A rivulet which can run straight to its destination is one thing; but a rivulet which is constantly twisting and turning about obstacles of all sorts can never attain the eroding effect of the first.

The author grants that, in periods of drought, springs and streams dry up more completely when a country is deforested, which is another way of saying that forests do, in a measure, regulate the run-off. As bearing upon this point, it was the writer's good fortune, about fifty years ago, to listen to a series of lectures by the noted geologist, Professor James Hall. He stated that, in his memory, the removal of the forests from large sections of the State of New York had resulted in drying up a large portion of the springs formerly so abundant. This was especially noticeable in the removal of the watering troughs which had been so numerous along the public roads. This coincides with the writer's memory as to Chemung County, in that State.

The next point controverted by the author is that forests regulate the flow by retaining snow, which slowly melts at a later period, and thus replenishes the streams. In reply to this: the writer well remembers that, in the woods on the hills near Elmira, N. Y., the drifts remained unmelted long after every bit of snow had disappeared from the cultivated portions of the valley. It was a common remark that there was enough left to add perceptibly to the flood usually expected in June. Exposure and the kind of forest may have had something to do with this, but the fact remains.

Passing to the third proposition—that forests increase the precipitation of moisture—it is not believed that this idea is now held to any extent by meteorologists. While admitting this, it does not follow that they have no effect upon storms. Probably no person would question the statement that a chain of mountains causes most of the moisture in the clouds to be deposited on their windward side, so that the rainfall is always greater there, and less on the opposite or leeward side. If this be true, are we not forced to believe that a belt of forest must have a similar effect, but in less degree?

When there is nothing to deflect them, the several moisture-laden currents of air tend to travel parallel to each other. Let them meet a forest, and the lower current is deflected upward, and, by mingling with the upper, causes the formation of clouds and rain.

The contention, then, is that forests tend to cause a more uniform

Mr. Collingwood.

distribution of the rainfall, lessening the mighty downpour and giving more frequent gentle showers; with this would follow fewer severe winds and an amelioration of the climate. It is probable, also, that forests act electrically to induce precipitation, by restoring equilibrium between earth and clouds.

The author's fifth conclusion in summing up is that "Soil erosion does not result from forest cutting in itself," and that "The natural growth which always follows the destruction of a forest is fully as effective in preventing erosion."

The writer's contention is: That forest destruction is always followed by a more rapid flood discharge, a consecutive increase in the flood volume, a diminution in the low-water volume, and a greatly increased erosion of the banks and damage of the flood plane in the lower reaches of every stream. His opinion is based largely on his life-long knowledge of a single river, he having been born in 1839 at Elmira, N. Y., on the banks of the Chemung. The conditions in his boyhood days were these: The stream had a fairly uniform width; the banks were largely grassed; the channel was comparatively uniform; there were very few bars, and all through the summer there was a good flow of water.*

On May 31st, 1889, and the following day, there was a freshet, exceeding in height any previously known, this being the result of a very heavy and extensive rain. The writer was employed to make an examination and report upon the best method of protecting the City of Elmira against such floods.

A careful survey was made of about three miles of the river, with cross-sections every 100 ft., these being carried entirely across the flooded portion of the valley.

Having lived elsewhere for about 20 years previous to this, the writer was amazed at the extent of the changes in the stream. At one point it had widened from about 600 to about 1 300 ft. All through this portion there were bars covered with black alder, and on the south bank there was a matted mass of alders, in some places 100 ft. wide. In early days there were no alders, and the land that had been cut away was rich and arable.

At other points, over considerable areas, every particle of mould had been washed away, down to the hard clay, and the whole region was covered with coarse weeds.

In a previous survey of another portion of the river, in a dry time, the stream practically disappeared in its gravelly bed. A gentleman living on the Susquehanna, at Wyalusing, told the writer that his lands along the river, which he used to consider the best parts of his

* This whole region, including the head-waters of the streams forming the Chemung, had been heavily wooded, and the cutting and rafting of pine lumber, for a market on the Lower Susquehanna, formed one of the chief industries for many years. Rafting ceased somewhere in the early Fifties, but lumbering and the cutting of hardwood for ties, fuel, etc., have continued the forest destruction.

farm, were now so cut and gashed by the freshets that they had largely lost their value. Any person riding along one of the railroad lines through the valley can see the damage done by the river, often at points considerably distant from its main channel. These devastations seem to be increasing, and records of the flood heights, kept for many years at Elmira, indicate a tendency of the floods to be higher as the years roll on. Mr. Collingwood.

To the writer's mind, the cause is very plain. One of the streams heading into the Chemung is very steep, and, when the heavy woods were cut from its bank, the flood-waters were more rapidly concentrated and poured into the main stream, so that the time required for the flood from a heavy rain to reach Elmira was very much shortened. The writer's recollection is that this time was reduced to one-half. The other two streams did their quota of damage, but not to the same extent. If this be true, the conclusion is irresistible: that the forests do act as powerful conservators and regulators of stream flow.

The conclusions seem sound from another point of view. Here is a stream flowing for centuries, having a uniform regimen, with fairly uniform section and smooth banks up to, say, 50 years ago, showing, up to that time, but few signs of destruction from floods. Within 20 years there is a complete change, and a beautiful stream is transformed into an irregular, wilful, destructive thing.

If the cutting of the timber did not do it, what was it?

THOMAS P. ROBERTS, ESQ.* (by letter).—This paper covers the subject in a very broad and comprehensive manner. Mr. Roberts.

It is a curious thing that, in relation to forests and stream flow, the river engineers seem to be arrayed as a small but compact body against "popular opinion." It must be remembered, however, that the engineers concern themselves only with large areas and navigable rivers, while the views of their opponents, in the great majority of cases, are based upon observations of restricted areas and small streams. It might be contended that a collation of the reports from a multitude of such local observers should be the most trustworthy basis for national legislation bearing upon the improvement of rivers, yet it may be questioned whether this conception is not an erroneous one.

In 1885 the writer was invited to address the American Forestry Congress at its Boston meeting upon a text similar to that adopted by the author. He was warned by the Secretary of the Congress that he was in a woful minority; in fact, almost a lone man, but nevertheless the Congress would be pleased to be entertained with his theories. His paper enunciated no theories, but simply presented certain records of navigable rivers, American and European, which appeared to show nothing in support of the idea that floods were increas-

* Civil Engineer, U. S. Engineer's Office, Pittsburg, Pa.

Mr. Roberts.

ing in height and frequency, or that the low stages had less discharge and were more protracted, on account of any works of man. The Congress was informed that perhaps the common belief that the whole was equal to the sum of all its parts was a proposition after all not without exception, as, for instance—to refer to extreme cases—the discharges of the Colorado and Rio Grande Rivers at times make up but fractions of the united volumes of their respective upper tributaries.

At a recent meeting, a speaker, after referring to the destruction of timber by the girdling process, leaving the trunks to decay uselessly then, rose to the climax of his argument, and said:

"Again, every man knows how in the late spring the roads in the open country may be firm and dry, but upon entering the forest one is sure to find pools of water and deep mud holes. This, gentlemen, illustrates the utility of forests, and it is the whole story in a nutshell: the open country dried up and the forests abounding with countless 'miniature reservoirs.'"

It was not the whole story, however, for, later in the season, in the same region, pedestrians might have been seen hastening toward the adjoining forests to escape drenching from the summer rain squalls. Still later, during the recent drought in Western Pennsylvania, beneath the shade of trees might have been seen corn actually dead for lack of moisture, while the corn 50 ft. distant, in the same rows in the open field, while somewhat wilted, still lived and filled its ears with a medium crop. There might also have been seen in the forests saplings here and there beneath larger trees with their leaves dried up and falling off. So great, apparently, was the draft of the tap roots of the big trees for water that no moisture whatever was left for a depth of several feet; otherwise the saplings would not have withered as they did.

It might be asked, is the forest, in its usual late winter and spring condition, with its "miniature reservoirs" full to overflowing, capable of holding back more water from later rains than the dry open country, as described by the speaker referred to? Or, when it comes to protracted droughts, are the forest areas contributing more water to the streams than equal areas of deforested country under similar conditions of soil composition, depth, and slopes? Writing in 1838, in his first report to the Monongahela Navigation Company, the late W. Milnor Roberts, Past-President, Am. Soc. C. E., the company's engineer, states, in reference to the drought of that season, that all the tributaries of the river between Brownsville and the mouth of the Youghiogheny River, a distance of about 42 miles, were dry at their mouths, and again the same thing occurred in 1895 and is practically true at this writing.*

* While it is true that the Upper Ohio, about October 1st, 1908, was lower than officially known since 1838, it is reported from East Liverpool, Ohio, that the "Indian picture writing" on a rock ledge at that place was not yet completely bared.

Mr. Roberts.

It appears that Colonel Chittenden must be correct in his assumption that, for the extremes of high floods and low water, as observed in our navigable rivers, forests exert no appreciable control. For the intermediate, or what might be termed summer, freshets, there is apparently some reason to think that forests have a measurable influence in reducing their height and prolonging their discharge, and this statement may hold, notwithstanding that less rainfall reaches the earth in forests than in open country. The fact is perhaps indicated for the Ohio in Table 2—for freshets not exceeding 10 ft.—and in Table 3—for the months of June and July—when the mean of freshets was 2 ft. higher in the second than in the first period. For the coal-boat stage—10 ft. or more—during the first period, the annual duration was 57.8 days; for the second period, it was 58.9 days. Doubtless the excessive rains in recent years will account for the difference in the higher stages. Speculations, however, regarding the Pittsburg river records in connection with forests, must be received with caution, in view of the fact that in eight of the counties on the Allegheny River water-shed, during the decade 1890-1900, there was a decline of more than 20 000 in the rural population, that is, exclusive of boroughs and villages, many one-time mountain farms being now covered with young trees or brush.

TABLE 2.—YEARLY MEAN NUMBER OF DAYS THE OHIO RIVER AT PITTSBURG REACHED THE INDICATED STAGES IN THE PERIOD SHOWN.

Years.	At or below 1 ft. stage.	Bet. 1 ft. -2 ft. stage.	Bet. 2 ft. -3 ft. stage.	Bet. 3 ft. -4 ft. stage.	Bet. 4 ft. -5 ft. stage.	Bet. 5 ft. -6 ft. stage.	Bet. 6 ft. -7 ft. stage.	Bet. 7 ft. -8 ft. stage.	Bet. 8 ft. -9 ft. stage.	Bet. 9 ft. -10 ft. stage.	Bet. 10 ft. -12 ft. stage.	Bet. 12 ft. -15 ft. stage.	Bet. 15 ft. -20 ft. stage.	Bet. 20 ft. -25 ft. stage.	Over 25 ft.
1855-1880	0.4	2.9	12.1	39.9	42.9	45.1	46.8	44.0	42.0	31.4	32.9	17.1	6.8	0.7	0.3
1881-1907	1.4	20.0	40.4	43.4	43.6	41.2	36.0	31.6	26.1	22.7	29.5	18.9	8.2	1.5	0.8

TABLE 3.—MEAN MONTHLY HIGHEST STAGES, IN FEET.

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1855-1880	14.71	13.99	16.02	15.85	12.00	8.65	7.26	7.13	7.44	6.91	10.85	14.97
1881-1907	17.07	16.76	18.26	15.45	11.44	10.52	9.18	7.80	5.71	6.75	10.34	13.66

Table 5 shows the number of rises in the Ohio at Pittsburg, by months, reaching to or exceeding 15 and 20-ft. depths, from 1855 to

Note for Table 2.—Some confusion in stages below 3 ft. since 1885—upon completion of the first dam in the Ohio—has resulted from attempts to conform the readings below the dam to the old Pittsburg datum; hence too much importance should not be attached to differences shown in the table for periods when the river was below 3 ft.

Note for Table 3.—But for a remarkable flood, reaching 31 ft. in September, 1861, the means of the two periods for that month would have been much the same.

Mr. Roberts.

1907, inclusive. The table shows that the winter storms of January, February, and March were much more frequent in the second than in the first half period. No forest influence appears to be indicated for the leafy months, a fact contrary to the common impression.

TABLE 4.—MEAN MONTHLY LOWEST STAGES, IN FEET.

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1855-1880.....	3.52	3.72	5.14	5.08	3.75	2.52	2.03	1.74	1.20	1.47	2.69	3.62
1881-1907.....	4.42	5.06	5.93	5.64	4.31	3.50	2.61	2.04	1.75	2.08	3.07	4.15

TABLE 5.

Period.	JAN.		FEB.		MAR.		APR.		NOV.		DEC.		MAY.		JUN.		JULY.		AUG.		SEPT.		OCT.	
	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.	15 ft.	20 ft.
First (1855-1881).	18	4	15	4	22	5	21	3	5	1	18	5	6	3	6	1	0	0	1	1	4	2	1	0
Second (1882-1907).	29	11	27	15	30	12	17	6	2	1	11	2	7	1	2	1	4	1	2	1	1	0	2	0

SUMMARY.

Period.	WINTER MONTHS.		LEAFY MONTHS.	
	15 ft.	20 ft.	15 ft.	20 ft.
First (1855-1881).	99	22	18	7
Second (1882-1907).	116	47	18	4

Since 1875 there has been no marked change in forest or brush-covered areas about the heads of the Ohio; while, during the winter and early spring months, covered in the second period, storms from the Gulf of Mexico, traversing the well-worn storm trail *via* Vicksburg, Chattanooga, and Charleston, W. Va., have, for some occult reason, reached Pittsburg with unprecedented frequency.

In the case of "cloudbursts," or storms of the tornado type, which, as a rule, traverse very narrow paths, few will dispute that a dense forest, by prolonging the discharge, even for brief periods, of the small streams they cross or follow, may reduce their flood heights. It is, however, from such limited areas, apt to be struck by violent storms

once in from 3 to 5 years only, that reports come that are made the basis of arguments applicable to large rivers. The worst known storm of this nature, which deluged some of the suburbs of Pittsburg in 1874, raising two small runs to 15 or 20 ft. depth and destroying more than 200 human lives, affected the depth of the Ohio only about 5 ft. Hence it becomes important to differentiate the storms producing floods on small streams and on navigable rivers, as Colonel Chittenden intimates is so necessary for a complete elucidation of the subject. Mr. Roberts.

In his treatment of co-ordinate topics, such as effects of forests on rainfall, dew, evaporation, erosion of slopes, silting up of rivers, and soil wash, Colonel Chittenden may be assailed by some who will differ with him in opinion. It is a mistake, however, to assume that those who endorse the author's views are antagonizing forest culture. No abler exposition, perhaps, of the importance and value of the forests has been presented than can be found in his résumé. He asks, however, that the new forests be planted convenient to the markets near the homes of the people. It is not wanton destruction on the part of the mountaineer to girdle the trees to make room for his corn and buckwheat crops, for gladly would he part with the biggest of the tree trunks for a pittance, but no offers are made him because the cost of transportation stands in the way.

So, while all of us have the same love for the trees, business considerations are very rude at times with our sentiments. We may hold on to all the trees, as the Indians did, without business; but, without continued business with the trees, we cannot make homes for many more modern Americans. Yet, without more of those trees to cut, what is to be done for homes in the future? The problem is indeed a vast one, well worthy the attention of the governments, National, State, and Municipal. Confronting us, however, there are other problems, not connected with the timber question, especially that of transportation on our inland waterways, but, so long as it is contended that transportation and the forest problem are in the same category, so long will the public mind be confused by conflicting statements and arguments, and so long will decisive action be delayed.

As for the silting up of our rivers, complaints regarding it do not emanate from navigators, at least on the Ohio. The writer, as a young man, lived for four years on an engineer steamer on that river in charge of the construction of dikes or wing-dams, surveys, etc., and upon inquiry has been assured that, if any changes in conditions of navigation have occurred during the past forty years, they have been for the better. A great rise from the Cumberland and Tennessee Rivers by backwater effect may cause the deposition of much silt in the main stream, noticeable at times for 200 miles or more, but eventually comes a current amply competent to restore the channel to its former condition. From end to end the great river has this reserve of energy.

Mr. Roberts. The theory to the effect that with afforesting of the regions about the head-waters there will result such clarification of the waters that the river will cease to be silt bearing, receives scant courtesy from the author. One writer argues that the sand now carried in suspension, acting like sand-paper, destroys the banks by "abrasive action." We have thus, in one breath, to consider the river banks being sand-papered down and the stream itself filling up. The Ohio between Pittsburg and Cincinnati has preserved its width between banks and its volumetric capacity very much the same for many decades, while, during the same period, that capacity has doubtless been largely exceeded by the volume of silt which the river has borne along. The author, therefore, is correct in saying that walling up the river banks would have no effect in reducing the volume of silt it transports; nothing would do that possibly for the Ohio but a tarpaulin spread over the steep hillsides of West Virginia and Kentucky, whence the greater proportion of the silt comes. Tributaries from soft-rock regions in those States can be named by the writer, from personal observation, which abounded with shifting sand-bars while yet the country about them was practically an unbroken forest. Those primeval forests did not prevent the carving of the nearly horizontal strata into the deep ravines found in the States named. If, however, the carving process is proceeding faster than formerly on some of the tributaries, troubles from it have not been reported on the main streams.

Regarding reservoirs to restrain floods on our Western rivers, the writer's opinions may have been perverted in youth by a parent who was in open opposition to Colonel Charles Ellet, the distinguished and able advocate of reservoirs. M. O. Leighton, Assoc. M. Am. Soc. C. E., has again raised the standard, but upon more specific and detailed lines than those of his great predecessor. It is a subject which should be considered apart from forest problems. Engineers must necessarily discuss its merits by weighing the cost with the advantages to be secured. If, for instance, it can be shown that levees along the Ohio can be built protecting all the property exposed to serious flood damages for one-fifth, or even less than one-fifth, of the cost of reservoirs necessary to secure that result, the engineers will be very apt to recommend the levee method. It requires no further surveys to hold tenable opinions on the questions herein involved. The matter of power development is, of course, a part of Mr. Leighton's project which should not be overlooked. At the same time we may have to content ourselves with fewer horses in our "power stables" than he hoped to provide. As to cost of flood-storage reservoirs in the Ohio Valley, readers are referred to an article* by Major H. C. Newcomer, Corps of Engineers, U. S. Army.

If Mr. Leighton's studies for the improvement of navigation do not

* *Engineering News*, October 8th, 1908.

meet with the results he expected, he can, in any event, congratulate himself upon showing the country where some very valuable water-powers can be found. The engineers fully appreciate the value of the elaborate topographic maps of the United States Geological Survey. Mr. Roberts.

Colonel Chittenden argues that the water-powers should be under the control of the authorities managing navigation interests. They are certainly related in the cases where the powers are located immediately at dams with locks, and it seems to be apparent that the shutting down or stoppage of flow in other cases, resulting in possible detriment to transportation interests, should be under the control of the navigation engineers.

The author's final argument that all the interests of forestry, power, and water transportation are within the purview or jurisdiction of the National Government will appear reasonable to most thinkers. The revenues of the individual States, and schemes of taxation, cannot well be adapted to projects of this nature, where State lines would so frequently stop an adequate exploitation. At the late Governors' Convention in Washington no strong voice was raised against this conception. What everybody wants in this country, one would think, should be constitutional, and in the few brief words of the preamble of the Federal Constitution our wise forefathers did not forget to say something about the promotion of the general welfare. Away then with the foreign criticism that our Constitution has caged a giant incapable of development in response to modern demands.

STEPHEN CHILD, M. AM. SOC. C. E. (by letter).—Colonel Chittenden's arguments are so evidently based on careful study and years of practical experience that they carry great weight, and certainly one who has been able to give these matters comparatively little investigation should hesitate to combat them. The writer's interest in them, however, is intense, and, if they remain uncontroverted, they will be a strong argument for politicians in Washington who are opposed to the Appalachian and White Mountain Forest Reserve bills. As a member of the Council of the American Civic Association, it was the writer's privilege, while in Washington, to use what little influence he had in the interests of these bills, the passage of which is believed to be of the utmost importance to New England and the South Atlantic States. Mr. Child.

The author takes up the influences of forests upon stream flow, and states "the commonly accepted opinion," "that forests have a beneficial influence on stream flow." He then proceeds to controvert these opinions pretty thoroughly. Certainly his arguments are strong and most interesting, and the writer frankly admits that he is entirely unable to meet them. As stated, the above-noted opinions are very commonly accepted at the present time, and no one is more responsible for their common acceptance in the United States than Gifford Pinchot, Assoc. Am. Soc. C. E., of the Forestry Bureau, whom the

Mr. Child. author frequently quotes and whose writings and lectures upon this subject have been accepted by many, up to the present time, as the final word on this subject. The writer, therefore, is very glad to know that Mr. Pinchot is to be given an opportunity to reply.

One of the author's statements, with reference to waste in lumbering operations, certainly appears to be extravagantly put. Commenting on the improved lumbering methods of the Weyerhauser Timber Company, he says:

"To speak of such timber as being 'lost' to the people, 'wasted,' and its acquisition as a 'looting of our heritage,' is as disingenuous as it is untrue. Will its lumber cost the consumer a cent more per thousand than if it were from a Government reserve? It is a wholly gratuitous assumption that our timber is going to be 'wasted' unless it is placed under Government control."

While the writer is in no position to controvert the statements as to the progressive methods now being used by the Weyerhauser Timber Company, he thinks he is well within the facts in stating that the improvements which this and other progressive lumbering concerns have adopted within recent years have been due largely to the campaign of education waged by Mr. Pinchot, and to the aroused public opinion which his work has brought about. It is hard to believe, as Colonel Chittenden would intimate, that there have not been in the past and are not now, in many sections of the United States, wasteful methods of lumbering; yet the statement quoted would imply that, and also that the Forestry Bureau, by its acquisition of Government reserves and its progressive handling thereof, has done, or can do, little good.

Colonel Chittenden leads up to the decision that if the people expect to have the Government purchase land for the rearing of forests for timber, or for the storage of water for any purpose other than navigation, or upon the theory that forests regulate stream flow and, therefore, are useful to navigation, the Constitution of the United States must be amended—a small matter, which he thinks can be easily accomplished in view of the widespread interest in these questions. The writer may be pardoned for disagreeing with him and for believing that, if his arguments are correct, he has furnished Speaker Cannon and the opposition to the Appalachian and White Mountain Forest Reserve bills a two-edged sword which they will not fail to use, and he speaks from the experience of personal contact with several members of the Committee on the Judiciary of the recent Congress. In the interests of these bills, therefore, and their great importance to an immense section of the country, the writer looks forward to Mr. Pinchot's reply with great pleasure.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is greatly impressed by the intrinsic value of this remarkably well written

paper. The author's remarks about Sir Gustav Wex's pamphlet on the flow of the Danube are well taken. In 1879 a second treatise was issued by the same authority, accompanied by elaborate tables and maps, and cross-sections of the river—all going to show the alleged fact that there is a decrease in the flow of water in springs, creeks, and rivers contemporaneously with an increase in the height of floods in cultivated countries. After most elaborate calculations he finally concludes that the decrease in rainfall due to this cause is found to be 0.0027 in. per annum, and the accumulated losses extending over 18 years, 0.0486 in. This refinement in the measurement of rainfall is unique. It is not at all surprising, therefore, that nearly all the meteorological stations in England, France, and Denmark, with records extending back from 100 to 120 years, failed to find any changes to speak of. Mr. Le Conte.

As far as the writer's observations extend, forests do not exercise any appreciable restraining influence on great river floods. The influence of the forest bed of vegetable humus is altogether too insignificant in amount to affect the problem.

The phenomenon of dew mentioned by the author is certainly most pronounced in the open, and at certain times of the year constitutes a very material addition to the precipitation. The writer recalls a striking experience on the San Joaquin Plains, California, while running a trial line, practically east and west, through the Town of Grayson. This was in August, when the rainfall was nothing. The line passed through 40 miles of continuous and unbroken wheat field. In places the wheat stood 42 in. high, was well headed, and, when harvested, frequently averaged 60 bushels to the acre. The surveying party began work early every morning without exception, and every member, the chainmen especially, was drenched with the heavy dew on the wheat, and never got dried out until noon each day. This continued for five days while passing through this mammoth wheat field. Of course, the popular phrase "dewfall" is a misnomer. The phenomenon is purely one of condensation; nevertheless, it is a very important and material element in the daily precipitation in the open, and should not be overlooked. In the forests, of course, this is *nil*.

Another most interesting feature about these plains is the effect of cultivation on the surface run-off. Experience everywhere shows that plowing the surface soil reduces the surface run-off enormously. The reason for this is easily understood when the physical facts are known. In the natural state the surface soil is generally sun-baked and hard. When the rains come, this surface crust sheds water like the roof of a house. On the contrary, when this land is plowed up into deep furrows, after the first rains have softened the ground, all the rainfall thereafter sinks in, and the run-off is practically nothing. More than that, the top soil, being broken up and porous,

Mr. Le Conte. stops subsequent surface evaporation enormously, and holds the ground-waters for the subsequent useful purposes of agriculture.

Of course, later in the season, when very heavy and prolonged rains come, the surface wash from the plowed lands is more heavily charged with sediment than it would have been before the lands were plowed. It should be remembered, however, that this run-off lasts for a much shorter time, and, therefore, it is an open question whether, considering the entire year, cultivation materially increases the total solids transported to the streams. The effect of cultivation, therefore, is to store the rain-waters in the soil, reduce surface evaporation, and enlarge and equalize the flow of ground-waters to the streams. These functions are certainly in keeping with the requirements of conservation, and tend to increase the percentage of annual run-off to rainfall. At all events, observations on lakes and artificial reservoirs certainly fail to show any noticeable increase in silting up due to cultivation of the soil.

The author's remarks about the relative efficiency of storage reservoirs in controlling the flood-water flow of rivers are exceedingly timely and strictly to the point. If these reservoirs are to be used for power or irrigation purposes, all efforts at flood control must be abandoned. The all-powerful interests of water-power and irrigation are diametrically opposed to storage capacity for the accommodation of great storm-waters, as these generally come at the end of the wet season. These two interests can never be reconciled, but will always be in endless conflict. In view of the large financial interests in water-power and irrigation, it is not difficult to see the final outcome of this conflict.

The big artificial reservoirs at the head-waters of the Mississippi are a great object lesson. They occupy ideal sites for storage reservoirs, and such sites are extremely hard to find.

The author mentions the fact that in times of heavy rainfall it sometimes happens that reservoirs will be compelled to discharge a greater quantity of water than would flow from the lakes in their natural state, that is, they would operate to increase the floods. Here again is evidence showing the conflict with power development, where the storage room is divided between several years instead of being confined to two years, as called for by the natural development. This conflict always arises, and it cannot be prevented. Necessity calls for it, and necessity knows no law.

The grand scheme of storage reservoirs proposed for the Ohio River system is altogether too Utopian, and is utterly impracticable as applied to the control of great river floods. Let water-power development and irrigation each stand on its own bottom, which both are amply able to do. The irreconcilable conflict between irrigation and low-water navigation in great rivers is rapidly approaching and will not down.

Mr. Le Conte.

The writer's sympathies are largely on the side of irrigation, simply because the public benefits derived therefrom are so much greater than the damages sustained by a few months' interruption of low-water navigation, which, in many rivers, does not amount to very much.

In the arid regions, where irrigation is the very life-blood of the entire community, it would be equivalent to public suicide to insist upon the rights of low-water navigation; the exigencies of the case forbid it. Here, it is well to call attention to a fact, not generally known, in reference to low-water navigation on the Ganges River in India. The head-works are located at Hurdwar, where the river leaves the foot-hills. The fall of the river at this site is from 10 to 12 ft. per mile. Below the dams the river bed is dry, the entire low-water flow being taken for irrigation. Going down stream, however, the irrigation seepage waters, together with underground flowage, increase enormously, and on reaching a point 40 or 50 miles below the dam the river is as wide and deep as it was before the head-works dam at Hurdwar was built. Later, in 1878, the engineers built a regular weir dam with locks for slack-water navigation, lower down the river at Narora—4 miles below the railroad bridge at Rajghat. This dam also takes all the low-water flow of the Ganges at this site, and necessarily leaves the river bed dry for a few miles; and then it is as wide and deep as ever. When one considers the enormous public benefits derived, viz., 1 600 000 acres irrigated on the Main Ganges Canal, and 1 900 000 acres on the Lower Ganges Canal, the small damage to navigation, caused by these great public works for a few months in the year only, sinks into insignificance and, in many cases, can be entirely ignored.

Mr. Maltby.

F. B. Maltby, M. Am. Soc. C. E. (by letter).—The presentation of this paper now, while the agitation and interest in the "Conservation of Natural Resources" is at its height, is particularly fortunate, and the placing on record, in an available form, of facts of observation rather than theories or popular opinions, is of the greatest value. The writer is in thorough sympathy with the movement for the preservation, and the intelligent and careful use, of forests and other resources of the United States, but agrees most emphatically with the author that the popular belief that deforestation is responsible for floods and low waters is an erroneous one.

There are ample reasons why forests should be carefully preserved and cared for and reforestation encouraged for the sake of future timber supply, without the necessity of charging to the destruction of the forests all the ills due to flood conditions on the one hand or the deficiency of water for industrial or domestic supplies and low stages for navigation during dry seasons on the other.

The author has shown in detail the fallacy of such beliefs, and these details show the painstaking care and thought that has been given

Mr. Maltby. to the preparation of the paper. The writer, unfortunately, has no other facts of observation to present as an addition to the discussion, and only wishes to approve most heartily the conclusions concerning the relations between run-off and deforestation.

During the Eighties or early Nineties, in the first days of extensive improvement work on the Mississippi by the General Government, it was impossible to secure appropriations for the construction of levees, except where built as an aid to the improvement of navigation, and one or more appropriation bills expressly prohibited such expenditures except as stated. There doubtless were engineers and others who honestly believed that the construction of levees did tend to improve conditions for navigation, and there is no desire to open up a discussion on this subject at present. Any one knowing the conditions and the vast benefits which have followed the construction of levees in the Mississippi Valley, however, will agree that there was ample and sufficient reason for their construction without any reference to navigation.

It is also true that the conservation of forests, the protection against flood conditions, or the improvement of low-water navigation are each of National importance; each can stand alone on its own merits, though it is undoubtedly true that all should be carefully considered by a commission charged with a careful study, in a broad way, of the whole subject.

The data presented by the author, as to the relation of stream flow and reservoir effect, are also particularly valuable in refuting some of the extravagant statements made by reservoir enthusiasts. It is to be regretted that the presentation of facts, with the calm arguments and conclusions based on them, cannot be given the same publicity as some of the wild assertions and schemes of the magazine writers. This cannot be expected, as there is nothing sensational in a consideration of the facts and conditions.

That much, very much, can and should be done in the direction of forestation, water supply for power purposes and navigation, and the control of floods, is undoubtedly true; and the engineering profession is to be congratulated on having the underlying facts concerning the relations between the various subjects so clearly brought before it.

Mr. Le Baron. J. Francis Le Baron, M. Am. Soc. C. E. (by letter).—This paper is very timely, and the author has handled the subject in a very thorough manner. His observations in regard to snow run-off from forest and open areas, are borne out by the writer's experience. In his early days, while running railroad lines in New England, in the early spring, he was often surprised to find the woods nearly devoid of snow, while deep drifts still remained under the road fences and stone walls, and in deep road cuts.

Mr. Le Baron.

The writer, however, must take exception to what he considers too sweeping a generalization of the statement that "there is really very little, theoretically, to support the claim that forests insure precipitation." It is undoubtedly true that when warm, moisture-laden air strikes a cold air current, the moisture will be precipitated in the form of rain. Forests are sometimes warm and sometimes cold. In northern latitudes forests are warm in winter and cool in summer; in the tropics, they are always cooler than the open, where the sun's rays are unimpeded. Take, for instance, the thick forests that clothe the eastern slopes of Nicaragua. For many months in the year, the warm trade-winds that have passed over the Caribbean Sea blow day after day from the coast over these forests, and the precipitation is phenomenal. It is common, when traveling on the larger rivers, to see spiral columns of vapor resembling the smoke of camp fires arising from the low hillsides when this warm wind from the sea is blowing over them. So perfect is the resemblance that at first the writer unhesitatingly supposed the vapor columns to be the smoke from the fire of some lonely hunter or settler.

There may be, as the author says, "very little to support the claim, theoretically"; but, practically, it is abundantly proven throughout the whole of that relatively narrow isthmus known as Central America, from Guatemala to Panama, as the writer has frequently observed. This phenomenon is particularly noticeable in Nicaragua, and the influence of the forest is there particularly observable, because that country is, in general, low, so that the excessive rainfall on the eastern side, amounting to 365 in. per year at Greytown, cannot be attributed to mountains. On the western side of that Republic, the precipitation is only about 45 in., and the forests are very different, being more open, and there are many large areas of cultivated lands and unwooded plains. All the moisture held by the eastern trade-winds is dropped when they pass over the dense and cool forest jungles that cover the entire eastern water-shed.

The author falls into another error of generalization, due, probably, to the fact that his observations have been confined to the limited area of the central and western United States. He states that "precipitation is nearly always greater upon the hills than upon the neighboring lowlands." This statement is quite true in British Columbia, where the observed rainfall in the Fraser River Valley is about 60 in., while less than 10 miles away, at Lake Coquitlam among the mountains, but in the same water-shed, the precipitation is 130 in. On the Central American Isthmus, however, the reverse is true. While the precipitation at Greytown is 365 in., and at Bluefields, about 290 in., at Tegucigalpa, Honduras, at an elevation of 3 000 ft. above the sea, it is about 90 in., and at Suina, Nicaragua, about 260 miles from the sea, it is about 100 in. Observations were taken for two years under the

Mr. Le Baron.

writer's immediate supervision, at Greytown and vicinity and near Suina, when he was in charge of the work of the Nicaragua Canal and other and later enterprises in that country.

Touching the fourth subdivision of this paper, the author says that "the increased erosion of the soil * * * does not result from forest cutting, but from cultivation." Cultivation, however, is a concomitant of deforestation; not the intensive cultivation of the market gardener on the outskirts of a great city, but the rough cultivation of the first settler. In the broad sense of the word, deforestation is not the cutting down of the trees alone, for, as the author remarks, the stumps and roots remain and a new growth succeeds, perhaps closer and more impermeable to water erosion than before. In the sense in which we speak of the deforestation of the United States, there is meant the complete eradication of the forest, not the cutting down of the timber alone, and this includes clearing and grubbing to eradicate the trees, root and branch. That is what has actually been done in the United States and in any country that is deforested. When this is done, the soil is broken and its subsequent cultivation is of secondary importance. The trees are gone, the roots and many of the stumps are removed, the soil has been broken, and, although the land may lie fallow and be uncultivated for several years, it is exposed to the wash of the rain. The effect of this wash is very noticeable on the hill slopes in Tennessee and Georgia, and even in New England, Ohio, and on all denuded hill slopes known to the writer.

In these localities the small brooks are filled with muddy water during every sharp rain. This was not so before the land was cleared, and is not so now in the adjoining woodlands.

The author states that "no possible development of forestry can increase the present percentage of forest-covered areas." This appears to be a very sweeping and unconsidered statement. To any one who has ridden over the vast prairies of the West, it is quite conceivable that Man, by irrigation or, in many places, solely by care and perseverance in judicious selection in tree planting, can and undoubtedly will in the next hundred years increase immensely the percentage of woodland, and there will still remain abundant open land for cultivation. At the same time, as the author states, many abandoned farms in the East will be grown up in wood. The writer knows of many such places in New England, and there are also many in Florida, and other Southern States, where large plantations have grown up in native forest trees since the Civil War, owing to inability to cultivate them profitably without slave labor.

The author states that the action of the forest to moderate the run-off and mitigate the severity of freshets fails entirely in heavy and long precipitation. He admits that the humus and débris in forests retain precipitation and act as a reservoir for a time, but eventually

may become saturated and thenceforth will be of no value for that purpose. In an extensive forest this impounding capacity is very great, and by just so much it reduces the peak of the run-off or at least the first part of the ascending curve, and there is no proof that, by the time the forest reservoir is full, the peak of the flood may not be past. Mr. Le Baron.

In his third proposition, the author say that "while small springs and rivulets may dry up more than formerly this is not true of the larger rivers." If there were no springs, rivulets, or brooks, there would be no rivers, and, if these all run dry, the large streams must run dry also.

The sixth proposition, that climate has not been appreciably modified by land clearing, may be true for large areas, in a general sense, but it has been modified, in small areas, in northern latitudes, by the added exposure to wind, so that the cold and inclemency are more pernicious for the farmer and stock grower.

In the author's discussion of the cost of hydro-electric power, using the storage reservoirs built by the Government for water conservation, he quotes an article by Mr. Henry D. Jackson, giving costs based upon averages of a number of different plants, compared with steam power.* Mr. Jackson's estimate includes the cost of the dam, "Building and works, steam, $10 000, water $77 000." The author does not take into account the cost of the dam and spillway, which are supposed to be already built by the Government for another purpose, so that this estimate, as it stands, is not applicable, but should be reduced by $67 000, or the cost of the dam, in favor of the hydro-electric plant. This makes a difference of 50% in first cost and the same in interest charge, and throws the balance in favor of the water power. Therefore, his subsequent deductions are valueless.

The writer quite agrees with the author in his remarks about diking for flood protection, and about transportation by river or rail. This question of the propriety of Congress aiding navigation, and having no authority, under the Constitution, to build power works to aid transportation by rail, only exemplifies the fact that the world is growing so fast that a Constitution, which was a perfect instrument in its day, can be outgrown by the totally changed conditions of the present time.

Edward P. North, M. Am. Soc. C. E.—This very able paper brings up a collateral and important question: Even if they accomplish all that is claimed for them, will National forests be a National beneficence? Mr. North.

The general and patent objections to the proposal now before the public are: The general installation of National forests would necessitate a still larger body of Government employees, and either tend to

* *Proceedings*, Am. Soc. C. E., October, 1908, p. 987.

Mr. North.

the perpetuation of a hierarchy, or to more or less violent changes of governmental policy.

Their control by the Central Government would impair our habits of individual initiative and also those of free association for production. These two habits, possibly more than any others, have been the governing factors in the progress of English-speaking peoples.

Their proposed locations, remote from centers of consumption for their products, by entailing the maximum cost for transportation, would diminish the resources of both producers and consumers. The fact that the General Government is a party does not obviate the necessity of making all expenditures return either insurance or profit.

As an alternative to the dedication of large areas to National forests under the control of the Central Government, and as free from the objections above recited, the following plan* seems preferable, namely, that lands covered by specified stands of trees should be free from taxation while the timber is growing and ripening. The numerous small groves which would be developed by the adoption of this measure would not be subject to the widespread destruction by fires invited by large forests, and, in addition to such modification of climate and run-off as may be due to timbered areas, would, by furnishing breeding places for insect- and vermin-eating birds, hinder, if not arrest, their extinction, besides adding to the beauty of the country; both additions not without pecuniary value. There should also be a provision of like kind for trees grown by the roadside. A prejudice against roadside trees, due to the combined effect of the trees and fences in forming drifts and keeping the roadbed wet, dominates the public mind long after the watering-cart has proved inadequate to keep down the dust or prevent the raveling of the roadbed.

Thinking that the illustration cited (with blotting paper) would have been more valuable if the paper had been surcharged—*e. g.*, with shavings—it is not contended that erosive and disastrous floods have not originated on uncleared areas, and they are likely to recur notwithstanding any development of forest culture. Reservoirs, if of sufficient capacity, seem to be valuable adjuncts, at least, to forests, in reducing the damage from all but exceptional floods. They would also be of great importance to more valuable National interests than either the growth of timber or the prevention of soil denudation. The most important of these interests is cheap transportation, with the production it develops. Following transportation closely, and governed by it, is manufacturing.

About forty years ago our Lake commerce was limited to vessels of 7- or 8-ft. draft. Excepting for a short time in the late Seventies, Lake channels have been persistently deepened and improved, so that the tonnage of vessels has increased faster than that of freight trains

* Proposed in *The Engineering Record*, June 13th, 1908.

on our railroads. In 1856, the year after the canal at the "Soo" was opened, the tonnage through it was 101 458. In 1907, 58 217 214 tons of freight passed through it. Mr. North.

Comparing the ton-mileage of our railroads with that through the "Soo," neglecting, of course, the through and local traffic not passing the locks, we have:

Ton-Mileage, etc., Reported for 1907.

Railroads as per "Poor."	233 137 507 807	0.782 cent.	$1 812 115 341
"Soo," as per "U. S. Statistical Abstract.".	48 221 465 547	0.080 cent.	38 457 345
	281 358 973 354		$1 850 572 686

The saving on this 17.1% of the total ton-mileage reported, compared with a like service by rail carriage, was more than $336 000 000. The total appropriations by the General Government for river and harbor improvements, with surveys, etc., pertaining thereto, were, up to March 2d, 1907, $552 943 025. If the ton-mileage of the Lakes not passing through the "Soo" was equal to two-thirds of that above recorded, the saving to the country by the low Lake freights was greater in 1907 than the total sums appropriated. Some will immediately object to this comparison: that the freight carried on the Lakes is of a lower classification than the average carried by railroads; but the lowering of freight rates during the season of Lake navigation has been recognized and fully understood for about thirty years, as also the increase in the high-class traffic on railroads.

In 1887, the first year of the admirable records kept by the U. S. Engineers at the "Soo," the ton-mileage through those locks was 4 458 544 804, which was carried at 0.230 cent, and the average railroad rate was 1.034 cents. The saving for that year was $36 026 200. If we average the 21 years we will have a rough aggregate of more than $4 000 000 000.

It would seem that a competent improvement of either the Lakes-to-Gulf waterway or of the proposed coastwise canal would result in a like increase of tonnage and an equal saving in the cost of transportation, for both routes are now offering much more traffic than was in sight or believed possible when the canal at the "Soo" was either projected or completed.

For nearly 20 years attention has been directed to the difference in growth of cities on the Lakes and in the valley of the Mississippi, where but immaterial improvements in depths have been made. And note is also taken that the progress of our centers of gravity of manufacturing values is following our centers of population, but about midway between the centers of population and the Lake front; which

Mr. North. shows, without doubt, that the cost of transportation is a very important factor in the location of manufacturing plants.

This influence is shown in particular by the nearly total withdrawal of heavy manufacturing from the New England States, where, from 1880 to the present, the freight rates have been about 50% higher than the average rate for the whole country. It is also shown by the ruin of the iron industries of Troy and Albany, with the depopulation of the counties about the head of navigation of the Hudson, consequent on the successful effort of the Chamber of Commerce of New York City to prevent the deepening of the river channel above Coxsackie.

The foregoing considerations have led to a growing insistance in the Mississippi Valley that the wealth of that section should be augmented by increasing the navigable depth of the river. The demand was growing too strong to be ignored when it was determined to combine any plan for improved navigation with one for "the conservation of our natural resources." This is of loss and damage to the prosperity of the country through the delay necessary for the consideration of so large and complicated a project. This delay will be prolonged by the selection of our most able opponent of internal improvements as Chairman of the Inland Waterways Commission, who, after the endorsement of this extended plan at Washington in May, 1908, is understood to have announced that there would be neither Congressional consideration of the Atlantic coastwise route nor appropriations for surveys in the Mississippi Valley. At the same time, the declaration for improvements of watercourses to a "standard depth" forecasts an intention, on the part of the protagonists of this movement for conservation, that the issue of their labors should be a standardization of what ought to be main arteries for the exchange of products to the economic depths of the smaller subsidiary streams. So that there will be no substantial reduction in the average of our total freight rates: that production may not be encouraged and the wealth of our lower Atlantic Coast and of the Mississippi Valley may not be unduly increased as the wealth along the Great Lakes has been.

The extension of our manufacturing interests, which in 1905 paid out in salaries and wages $3 186 310 754 and produced values of $14 802 147 087, as well as the enlargement of the area of cheap freightage, is threatened by this conservation plan. It is understood that the Central Government proposes to seize all sources of water-power and put them under bureaucratic control. This control may be relied upon to hold the value of these natural resources so near a prohibitive price and to hamper their use by vexations restrictions in such a way as to retard unnaturally their development. All the employees of this branch also are likely to be auxiliary to Government by executive decree.

Mr. North.

The development of water-powers, and particularly of our timber supply, without control by the Central Government, would free the country from the danger to be apprehended, under our elective system of large bodies of Government employees who could be made to act with nearly as much unanimity as a regiment of soldiers. The individual growers of timber, with their employees, could not be induced to act together. The lands selected would be chosen on economic grounds, without taint of political corruption or even political influence. They would be widely disseminated, mostly on the broken ground heading all our streams that do not originate in swamps, and the undisputed cooling effect of groves would break up the ascending currents from highly heated areas which dissipate possible rain clouds. The lumber they furnish would be in proximity to the demand, while the waste products of scattered groves would have a local use and value probably more than offsetting the economies of manufacture in large establishments. In both cases, the areas devoted to tree growing would not contribute directly to local taxation; but, in the case of woodlands cultivated by individual initiative, a careful selection would be made of the less productive and less easily tilled areas. Our proposed National forests, on the contrary, would include in their large areas all those fertile spots that support farmers, and cover the land with men increasingly trained to look to the Government for their means of support.

This entire scheme of Conservation of National Resources seems to be on a parity with the present demagogic demand that no more railroads should be built lest there should be ruinous competition, coupled with the claim that the return on the capital invested in railroads should be carefully limited through the supervision of our Central Government instead of by the action of free competition. It seems to be planned that even our waterways—if improved—shall not compete freely.

Our best National resources are the production and consumption of our people. It is doubtful if our entire natural wealth would have sold for £5 000 when Elizabeth and Raleigh commenced exploiting this country. Until natural resources enter into destructive consumption, few of them have anything but a speculative value. Some of these resources are destroyed by their use. The power of falling water, though destroyed by that act, renews itself indefinitely, and any nonuse of that power is an irremediable loss to the world's wealth. There is no impairment of value to any watercourse by its use, and, when there is money enough available for its improvement, neglect of its possible aid to production seems to be an economic crime.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS FITCH ROWLAND, Hon. M. Am. Soc. C. E.*

DIED DECEMBER 13TH, 1907.

The name of Mr. Rowland will be, each year, recalled to the Society when the award of the Thomas Fitch Rowland Prize is announced at the Annual Meeting. The prize was originally instituted by the Society at the Annual Meeting of 1882, in recognition of the generous action of Mr. Rowland in connection with the fund created for assuring to the Society the permanent ownership of a building suitable for its needs and its growth. Mr. Rowland did not know that the institution of this prize was contemplated, and, when he was informed that it had been done, he insisted upon giving to the Society the amount, the annual interest of which now provides the prize. At the request of the Board of Direction, he suggested the conditions for the award, namely, that the paper for which the prize is given shall describe accomplished works of construction, state their cost, and especially note any errors, in either design or execution, which may have developed during construction. His long and varied experience in construction led to the desire that engineers connected with the progress of works should be encouraged to describe, not only the finished results, but also those changes and modifications which might occur during their progress, and from which he believed much useful knowledge would be given for guidance in other and future work.

Thomas Fitch Rowland was born in New Haven, Connecticut, on March 15th, 1831. He was the son of George Rowland and Ruth Caroline Attwater. He was a lineal descendant of the Honorable Thomas Fitch, the last colonial governor of Connecticut.

He was educated at the public schools of New Haven. At an early age he acted as miller's boy in his father's grist mill. This mill was demolished upon the construction of the New York, New Haven and Hartford Railroad, and he became the first apprentice in the machine shop of that road. While in that employ he was detailed for, and acted as, fireman of the locomotive on the third passenger train which ran from New Haven to New York.

In 1850 Mr. Rowland left the railroad and became Second Assistant Engineer of the *Connecticut*, a large side-wheel steamboat, carrying passengers and freight between Hartford and New York.

* Memoir prepared by the following Committee: John Bogart, George S. Greene, Jr., and Edward P. North, Members, Am. Soc. C. E.

His next service was with the Allaire Works of New York, large builders of steam engines. While with them he designed the engines of the U. S. revenue steamer, *Harriet Lane.* He then entered the Morgan Iron Works, owned and operated by George W. Quintard. He made the designs, specifications, and estimates of cost of the machinery for the U. S. gunboat, *Seminole,* and was put in charge of the construction of that machinery. He then took charge as Superintendent of Construction of an iron side-wheel steamboat which Mr. Quintard built at Greenpoint, Long Island, for use at New Orleans. Associated with him in that work was Mr. Samuel Sneden, a prominent builder of wooden vessels. He and Mr. Rowland then formed a partnership for building iron and wooden steamers, and general structural work. The first contract obtained by this firm was the construction, for the Croton Aqueduct Department of New York City, of a wrought-iron water pipe 7½ ft. in diameter, about ¼ mile in length, and located on the top of the High Bridge over the Harlem River. Another early contract was for an iron vessel designed by the late John Ericsson, Hon. M. Am. Soc. C. E. This firm also built the hulls for the steamboats, *Continental,* the *City of Boston,* and the *City of New York.* Mr. Sneden left the firm in 1860, and Mr. Rowland thereafter continued the enterprise under the name of The Continental Iron Works. Early in 1861 he built a number of naval gun carriages. He also built twenty revolving mortar-bed carriages for the Navy Department. These, with their mounted mortars, were installed on a number of vessels which, during the Civil War, were known as the Porter Mortar Fleet.

In 1861, for John Ericsson and associates, Mr. Rowland built the iron-clad, single-turret, floating battery, at first called Ericsson's battery, but directly designated the *Monitor.* This was the original *Monitor* which arrived in Hampton Roads during the night of the day on which the Confederate iron-clad, *Merrimac,* had sunk the U. S. frigate, *Cumberland,* had driven ashore the U. S. frigate, *Congress,* which was entirely burned, and which doubtless during the next day would have captured or destroyed the frigates, *Minnesota* and *Roanoke,* and other smaller vessels, and thus opened the way to Northern harbors. The struggle during the next day between the great iron-clad, *Merrimac,* and the comparatively diminutive *Monitor,* and the final victory of the *Monitor,* are notable events in naval history, which determined new departures in naval construction and armament. The speed as well as the thoroughness of construction of this original *Monitor* was notable, particularly as the vessel differed in nearly every feature from anything previously built.

Mr. Rowland immediately proceeded with the construction of the monitors, *Montauk, Kaatskill,* and *Passaic,* and the double-turreted monitor *Onondaga.* He also built the light-draft gunboats, *Cohoes* and *Muskoota.*

In 1870 he built the ferry-boats, *Fulton,* and *Farragut,* and afterward the *Atlantic,* and the *Brooklyn,* all for service between New York and Brooklyn.

For many years Mr. Rowland devoted great attention to the design of gas-manufacturing installations, and he built a large number of these in various parts of the United States. Among the very large ones are those of the Commercial Point Station of the Boston Gas Company, and the Twenty-first Street, the Forty-fourth Street and the Ninety-ninth Street Stations of the Consolidated Gas Company of New York. The Fourteenth Street gas-holder and steel tank for the same Company, also designed and built by him, was, at the time of its construction, the largest of its kind in the United States and a notable achievement in gas engineering.

In 1887, his business was incorporated as The Continental Iron Works, Mr. Rowland being its President up to the time of his death.

For many years Mr. Rowland had devoted much study to the perfection of the art of welding iron and steel plates in various forms and shapes, and, about 1887, he designed the process and the requisite machinery for the manufacture of the Fox corrugated and the Morison suspension furnaces now used in the well-known internal type of boiler.

In 1855 Mr. Rowland married Mary Elizabeth Bradley, of New Haven, who died in March, 1902. He is survived by two sons, Thomas F. and Charles B. They have been associated with him in The Continental Iron Works, and both of them are Members of the Society.

Mr. Rowland continued in active touch with his works until within a few months of his death, although he had been in failing health for several years. He was a devoted member of the Episcopal church, having joined it in early life. He had many charitable, philanthropic, and historical interests. Although he had to do with such important works, during a long and remarkably active life, he was always of a modest character and retiring disposition.

He took the greatest interest in the welfare and progress of the American Society of Civil Engineers. During the periods of its growth, when many perplexing problems were encountered by its officers, they could, and always did, receive from him substantial aid and well-considered and judicious advice.

Mr. Rowland was elected a Member of the American Society of Civil Engineers on December 4th, 1867, and was made an Honorary Member on December 20th, 1899. He served as a Director in 1871, 1872, and 1873, and as Vice-President in 1886 and 1887.

STEPHEN ARNOLD MITCHELL, M. Am. Soc. C. E.*

DIED MAY 21ST, 1908.

Stephen Arnold Mitchell was born at Point Pleasant, Ohio, in 1863. He was of Scotch-Irish parentage, his ancestors being among the early settlers of the State of Maryland. In 1868 he came to Kansas City, Missouri, where he received his early education, being graduated at the age of sixteen from the Central High School. In 1883, he was graduated from the Missouri State University, receiving the degrees of B. S. and C. E.

Immediately after graduation Mr. Mitchell commenced the practice of his profession under Messrs. Knight and Bontecou, Civil Engineers, in the location and construction of the Kansas City Belt Railroad. After the completion of this work he spent one year in the Engineering Department of Kansas City, confining his attention chiefly to the construction of sewers and paving. For about six years, from 1886 to 1892, he devoted himself to the general practice of his profession, making surveys and soundings to guide in the location of the Chicago, Milwaukee and St. Paul Railroad Bridge across the Missouri River at Randolph, Missouri, and assisting in the construction of a line of railroad connecting that bridge with the Kansas City Belt Line Railroad. Later, he associated himself with Mr. John W. Neir and an older brother, under the firm name of Mitchell and Neir, and for several years was very active in the design and construction of water-works in many towns of the rapidly developing State of Kansas, in Fort Payne, Alabama, and in Elwood, Indiana.

From 1892 to 1901 Mr. Mitchell was Assistant City Engineer of Kansas City, Missouri, serving the city with a degree of fidelity and self-sacrifice that won for him numerous friends and many staunch supporters regardless of political affiliations. He left the service of the city from choice, well equipped by experience to assume weightier and more independent responsibilities. So great was the confidence he had inspired by his faithful service to Kansas City that his advice was frequently sought in all matters pertaining to municipal engineering. He assisted in placing a value upon the water-works property which Kansas City purchased from the National Water Company, and later became one of the members of a commission to devise extensive and comprehensive improvements in this system, acting for several years as Chief Engineer of Construction in carrying out the plans of the Commission.

In 1906, Mr. Mitchell planned more than 50 miles of sewers and a complete system of street improvements for Fort Smith, Arkansas, a

* Memoir prepared by Wynkoop Kiersted and Alexander Maitland, Jr., Members Am. Soc. C. E.

city in which he had filled the important position of Consulting Engineer to the Board of Improvements for some time. He died on May 21st, 1908, at the age of forty-five. At the time of his death he was in personal charge of the construction of the public works he had designed for Fort Smith, and accordingly had taken up his residence in that city. He is survived by his wife.

Mr. Mitchell's devotion to his professional work, his uniformly genial disposition, and his profound integrity, won for him many life-long friends who mourn his death and feel keenly the loss of his good fellowship.

Mr. Mitchell was elected a Member of the American Society of Civil Engineers on October 2d, 1907.

GEORGE W. RAFTER, M. Am. Soc. C. E.*

DIED DECEMBER 29TH, 1907.

George W. Rafter was born in Orleans, Ontario County, New York, on December 9th, 1851, being the youngest of five children. His mother was Eleanor Willson, whose grandfather, James Willson, was identified with the Revolutionary history of Cherry Valley, New York. In 1739 he assisted in making the first survey of the region around Cherry Valley, for which he was given a tract of land on which his descendants still live. On his father's side, Mr. Rafter was of Scotch-Irish extraction, his great-great-grandfather Rafter having emigrated from the north of Ireland to Northumberland County, Pennsylvania, about 1750.

Mr. Rafter's father owned a developed water-power at Orleans, which operated a flouring mill and other manufacturing establishments, from which he made a comfortable living for the family and accumulated some property. Possibly the experience with that water-power in his boyhood was the influence which inclined Mr. Rafter toward the improving of water-powers in general by water storage, in later years (because, after his father's death, which occurred when Mr. Rafter was seven years old, this water-power was sold several times, but reverted back to his mother because of the inability of the purchasers to use it efficiently on account of the variation in the flow caused by the removal of forests, etc.).

That Mr. Rafter had an inherited taste for his chosen profession is indicated by the fact that he had, in his library, a book on mathematics printed in London in 1704, which his ancestor, James Willson, had brought with him when he came to America. An interesting incident in connection with this book is the fact that when James Willson was

* Memoir prepared by J. Y. McClintock, M. Am. Soc. C. E.

moving his family from Albany to Cherry Valley, by flatboat up the Mohawk, the chest in which it was contained slipped off into the river. The spot was marked, and the following spring the chest was recovered, but the book shows very plainly that it has been water-soaked.

Mr. Rafter was a prominent representative of the sturdy class of civil engineers who have had a controlling influence in shaping the development of this wonderful country on a scale of such magnitude as to be beyond the dream of our ancestors. He assisted materially in advancing the application of engineering in many directions. As a boy he attended the public schools, and later spent three years at the old Canandaigua Academy where his roommate and chum was Lyman E. Cooley, M. Am. Soc. C. E., and where his associates included many other men prominent in after-life. On leaving the Academy, he went to Cornell University.

In 1872, Mr. Rafter married Miss Alyda Kirk, and they were a mutual help and comfort to each other for thirty-five years, bringing up to maturity two daughters, Ethel, now the wife of Dr. John R. Williams, of Rochester, New York, and Myra Willson, who, for several years past, has been an efficient aid to her father, assisting him as stenographer and computer in the preparation of his important reports. Mrs. Rafter died in May, 1907, and, after her death, Mr. Rafter was oppressed with lonesomeness, and, to a large extent, lost interest in his profession, so that, to those who knew him intimately, it seemed not unfitting that he should follow her so closely.

Beginning his engineering work in 1873, Mr. Rafter took a position in the City Surveyor's office at Rochester, where part of his time was occupied. At the same time he studied architecture in the office of A. J. Warner, with Ex-Mayor James G. Cutler as a fellow student. At this period he also found time to teach mathematics in a private school conducted by Professor Reed, where he made lasting friendships with boys who have since developed into the leading men of the community, such as the Honorable James Breck Perkins, Wheelock Rider, M. D., and others.

In 1876, he was Assistant Engineer on the Rochester Water-Works, while in 1877-78 he was in private practice as a civil engineer, filling, among other engagements, that of Engineer to the Rochester and Lake Ontario Railway Company. In 1880-82 he was Engineer on the construction of the Texas Pacific and Missouri Pacific Railways, having charge, in 1881, of the water supply of the Texas Pacific Railway across the "Staked Plains." In 1882-83 he was in charge of the construction of water-works at Fort Worth, Texas.

Mr. Rafter was Assistant Engineer of the Rochester Water-Works in 1883-87, and also Engineer of the water-works at Fredonia, New York. In 1883 he made a survey of Honeoye Lake with reference to converting it into a storage reservoir for the benefit of the Rochester

mills. This was the beginning of the great work done by Mr. Rafter, in the study of the possibilities of regulating the Genesee River, which afterward occupied so much of his attention.

From June, 1888, to October, 1890, he was in charge of the additional water supply for the City of Rochester, and was Acting Chief Engineer of the Rochester Water-Works in 1890. During these years he showed how he had developed into a strong man. The city approached the limit of the capacity of the water-works, and the then Chief Engineer had advanced a project with unusual features, causing the business men of Rochester to doubt the soundness of his judgment, so that they even questioned the necessity of an additional supply, on the ground that the consumption per capita was beyond reason if the existing conduit was delivering the quantity of water for which it was designed. They insisted that, as a preliminary, means should be taken to gauge the quantity of water delivered to the city, and that, with proper care, shortage of water might be prevented. The Chief Engineer asserted that it would be impossible to measure the water without causing a water famine, and that, under the conditions, it would be impossible to prevent a shortage of water to the citizens. A committee of the Chamber of Commerce induced Mr. Rafter to say that, if he had the responsibility, the water could be measured and a water famine avoided. The Committee took him at his word, and, upon its insistence, he was made Acting Chief Engineer. While occupying this position, he laid the City of Rochester under lasting obligation by introducing and operating an ingenious and unique scheme of throttling down certain districts, so that the insufficient supply was distributed in such a way as to preserve the public health and reduce discomfort to the lowest possible degree.

The following years were occupied with varied work, including sewage disposal at Albion and Holley, New York, and the West Virginia State Hospital for the Insane; water-works at Berwick and Nescopeck, Pennsylvania; sewage disposal works at Lawrenceville, and Geneva, New York; and Consulting Engineer to the Warsaw, New York, Water Company. In conjunction with Mr. M. N. Baker, Mr. Rafter prepared the exhaustive and comprehensive work, "Sewage Disposal in the United States," which has since been a standard textbook, and of great importance, because it was the first complete exposition, in the United States, of a subject then becoming of such vital importance and interest. It was dedicated by the authors "To the Health and Prosperity of American Cities and Towns," and there is no doubt that many such cities are under obligation to Messrs. Rafter and Baker for the information contained in this manual.

For some time Mr. Rafter had been preparing himself by studies with the microscope, and, while acting as Sanitary Expert to the Boston Water-Works, he, with Professor W. T. Sedgwick, devised the Sedgwick-

Rafter method of water analysis, well known to biologists. Mr. Rafter was one of the pioneers in making practical use of the application of biology to city water supplies. He prepared several papers on this subject, including: "On the Fresh Water Algæ and their Relation to the Purity of Public Water Supplies";* "How to Study the Biology of a Water Supply"; "On Some Recent Advances in Water Analysis, and the Use of the Microscope for the Detection of Sewage Contamination"; "On the Micro-Organisms in Hemlock Water"; "Microscopical Examination of Potable Water"; and, together with Doctors Mallory and Line, a pamphlet entitled "On *Volvox Globator* as the Cause of the Fishy Taste and Odor of the Hemlock Lake Water in 1888."

In 1893 Mr. Rafter began work for the State of New York on the subject of controlling the flow of rivers and the construction of storage reservoirs, and, in connection with it, a comprehensive study of the hydrology of the State. His first work in this line was on the Genesee River, and consisted in making exhaustive surveys and borings, and designs for a great dam just above Mount Morris, at a site previously established by the State Engineer. In connection with this work, elaborate tests were made of the strength of concrete. In this study, he arrived at the conclusion that it would be more advantageous to locate the dam above the High Falls at Portage instead of at Mount Morris. A further appropriation having been made by the State, he was able, in the following years, to verify his conclusions by careful surveys, and to determine that, for a less expenditure, a reservoir of double the capacity could be created which, on account of the additional fall of 500 ft. available, would double the horse-power. Thus he fixed the plan of regulating the river which will be ultimately carried out.

During these years of work on the Genesee, with the aid of the Rochester Chamber of Commerce, many plans for carrying out the project were tried and many schemes were presented to the Legislature. Finally, at the suggestion of the Governor, who had vetoed a bill which provided for the work being commenced by the State, the Genesee River Company was organized, and legislative authority was secured for it to build the works. Mr. Rafter, more than any one else, worked out the details of these various campaigns and enlisted the capital necessary for such a company to begin operations. It was one of his disappointments that, because of the impossibility of securing the consent of the larger owners of water-power in Rochester, the company was unable to carry out the work begun by it.

He also made surveys for the regulation of the flow of the Hudson River by the establishment of reservoirs on its head-waters, and designed and superintended the construction of a great dam on Indian River, a branch of the Upper Hudson. He likewise planned several additional reservoirs, including the great one at Schroon Lake.

* *Transactions*, Am. Soc. C. E., Vol. XXI, p. 483.

One result of these reports on the various rivers was the formation, in 1902, of the New York State Water Storage Commission, of which Mr. Rafter was a member. This Commission made a very valuable report, and recommended a law establishing a River Improvement Commission, to which power should be given for regulating any river in the State. This was adopted by the Legislature, in a modified form, and under it work was begun on several streams. Later, the River Improvement Commission was combined with the Water Supply Commission, which is now proceeding in a comprehensive manner to develop the natural resources of New York State.

In 1894, Mr. Rafter was sent abroad by the State Engineer of New York to study movable bridges in Europe, and at that time he spent several months in England, Holland, Germany, Italy, and France, on work relating to this subject. He also investigated several high masonry dams, and, upon his return, made reports to the State Engineer on each of these subjects.

In 1898-99, Mr. Rafter had charge of the water-supply investigation for the United States Board of Engineers on Deep Waterways. Elaborate surveys and estimates were made for a ship canal at Oswego, from Lake Ontario through Oneida Lake and down the Mohawk River to the Hudson. This work was, perhaps, the most important of his undertakings, because it included a plan for the establishment of great reservoirs on the Black and Salmon Rivers, a water-supply canal of more than 90 miles in length, or an alternate tunnel some 28 miles in length, and also a thorough examination of the applicability of hydraulic formulas to heavy flows over weirs, and a set of experiments on a large scale far beyond anything in that line available at that time.

Since 1900 Mr. Rafter had been in private practice as a Consulting Engineer, during which time he made valuable reports on a variety of projects. In 1904, he prepared, for the State Geologist of New York, a report on the hydrology of the State, which is a recognized authority on matters relating to stream flow.

Mr. Rafter was largely instrumental in inducing the beginning of a systematic observation and record of stream flow, both by the State of New York and the United States Government, and his work on "The Hydrology of the State of New York" will be ever recognized as an important foundation for the great studies which have been begun looking toward the development of the immense national wealth to be continuously created by the streams of New York. He wrote five papers, published in the Water Supply and Irrigation Series issued by the United States Geological Survey, and, in the very complete engineering library which he had collected, were included fully 175 books and papers of which he was the sole or joint author. He was a very prolific writer, and, in addition to the books and papers which have been published, he had written innumerable professional reports.

Mr. Rafter was a good citizen, in the best sense of the word—he gave of his knowledge and experience freely for the benefit of Rochester, where he had raised the family he loved. As a member of the Chamber of Commerce, from its organization, he was ever ready and active in working for the best interests of the community. The writer knew him personally for twenty-five years, and saw a great deal of him, having worked with him on many occasions.

Mr. Rafter was an indefatigable worker, and, on any subject which interested him, he would strive to know it to its farthest detail. His brain was big, and contained a vast store of useful and entertaining information. Sometimes he was gruff, but his heart was warm, and he had a comprehensive regard for others, perhaps more especially for young engineers. He had a horror of their becoming what he termed "lackadaisical," and sometimes his advice to them was strongly worded, but he was always anxious to explain and demonstrate when they were receptive. For several months, during the work on the Deep Waterways surveys, the writer was closely associated with Mr. Rafter, and enjoyed discussing with him all kinds of professional, religious, and other questions, and, for hours, he would recite from the most beautiful masterpieces in literature. He was a rather large man, physically, and the writer sometimes likened him to Samuel Johnson, whom he certainly compared with in capacity of brain. His voice was deep and musical, and sometimes his feelings were touched so that his recitations were most eloquent.

Mr. Rafter left his home in Rochester, New York, on December 1st, 1907, expecting to spend the winter in Europe looking up matters in which he was interested. While in Karlsbad, he contracted pleurisy, and died in two days.

Mr. Rafter was elected a Member of the American Society of Civil Engineers on April 2d, 1884.

CLARENCE GEORGE VAUGHN, M. Am. Soc. C. E.*

DIED AUGUST 4TH, 1908.

Clarence George Vaughn was born in Geauga County, Ohio, on September 25th, 1853. He was the son of G. W. Vaughn, M. Am. Soc. C. E.

At the age of eighteen years, Mr. Vaughn was employed in the Engineer Corps of the Kansas Central Railroad in the humble position of Axeman, but was rapidly promoted to Rodman, Levelman, and Instrumentman. Subsequently, he accepted the position of Assistant

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

Engineer of the Kansas City, Wyandotte and Northwestern Railroad at Independence, Missouri, and, later, was employed by the Denver and Rio Grande Railroad as Assistant to the Division Engineer on the Leadville and Wheeler Branch. He was transferred to Poncha Springs, on the line from Salida to Gunnison, as Division Engineer, in 1880, and resigned shortly afterward to enter the service of the Mexican Central Railroad in the City of Mexico. From 1882 to 1885 he did miscellaneous work in Kansas City and vicinity.

From 1885 to 1889, Mr. Vaughn was in the employ of the Chicago, Milwaukee and St. Paul Railway, as Assistant Engineer, doing special work both in the office and in the field. His health began to fail at this time, and, for some months, he was incapacitated for work. He finally recovered and, in 1890, entered the service of the New Orleans and North Western Railroad as Assistant Chief Engineer. He became Chief Engineer in 1893, which position he held until 1904, meantime also undertaking, in 1901, the position of Chief Engineer of the Eldorado and Bastrop Railroad.

In 1905, Mr. Vaughn did business as Consulting Engineer at Natchez, Mississippi, but evidently could not resist the fascination of railroad engineering, for in December, 1905, he took charge of the construction of the Southern Pacific Railroad Company's line from Lafayette to Port Allen, Louisiana, as Construction Engineer, in which position he served until August, 1907, when he went to his Natchez home in an unsuccessful attempt to recover his health. He gradually grew weaker and, after suffering for a year, entered into rest on August 4th, 1908.

The greater part of the last eighteen years of Mr. Vaughn's life had been spent in the swamps of Louisiana and Arkansas, which no doubt contributed to his ill-health and tended to shorten his useful life.

Mr. Vaughn was an accomplished engineer, and was a gentleman of the highest possible type, a veritable Chevalier de Bayard, "without fear and without reproach." His disposition was modest and retiring, and his thoughtfulness of the comfort and feelings of those about him was ever a source of wonder and pleasure to them. It is not fulsome praise to say that it was a genuine privilege to have known him, and that no man could know him well without being a better man for it.

He is survived by his father, G. W. Vaughn, M. Am. Soc. C. E., by a son who is practicing medicine in Kansas, and by his widow who resides in Natchez, Mississippi, whose grief is shared by hosts of friends, for truly "none knew him but to love him."

Mr. Vaughn was elected a Member of the American Society of Civil Engineers on November 7th, 1906.

620.6 William P. Morse

AMERICAN SOCIETY

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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THE PURIFICATION OF GROUND-WATERS CONTAINING IRON AND MANGANESE.

By Robert Spurr Weston, Assoc. M. Am. Soc. C. E.

To be Presented February 3d, 1909.

Introduction.

The primary source of ground-water is the moisture in the air. A part of this moisture, which precipitates as rain, snow, or dew, soaks into the ground until it reaches an impenetrable stratum where it collects in greater or less amount and is called ground-water. The soil air also contains moisture, a part of which condenses beneath the surface. Although Aristotle considered the mountains of the earth as sponges,* whose interstices collected the rain which appeared later in springs and wells, the present theory of the circulation of meteoric water is comparatively modern. Even as late as the middle of the seventeenth century, many of the philosophers believed that the source of ground-water was the water of the sea and of rivers which, soaking into the lower strata of the earth, was evaporated and then condensed nearer the surface.

The term "ground-water" comprises spring water and water from shallow, deep, and artesian wells. From an engineering standpoint,

*F. R. Koenig, *Schillings Journal für Gasbeleuchtung und Wasserversorgung* 49, 1033.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

these waters differ in their hydrology only. They have the same primary source, and their physical and chemical characteristics differ in degree rather than in kind.

In the literature of the most ancient historic peoples there is frequent commendation of ground-water. It was especially attractive to those living in warm climates. Ground-water, especially flowing well water, "fountain of living water," was frequently used as a metaphor for expressing excellence; likewise, the modern conception of a satisfactory supply is best illustrated by a good ground-water, that is, one which is perfect in appearance, agreeable to the taste, soft, and bacteria-free. Furthermore, the object of all schemes for supplying water from other sources is to approach the characteristics of such a ground-water as nearly as practicable. Such waters as the Alpine supplies of certain European cities and the well-water supplies of a few New England and Atlantic Coast towns are good examples of this kind of water, of what from a laboratory standpoint is "pure spring water," the popular standard of excellence.

It is quite unnecessary to call to mind how satisfactory are such supplies. The examples are numerous, especially in Europe, and communities supplied with such waters are unusually free from epidemics of water-borne diseases. In Leipsic, for example, it is said that surgeons when performing operations use water direct from the tap as they would sterilized water.

Most of the best of the large water supplies in the United States are from upland surface sources, but the increase in population on the catchment areas is imposing such additional burdens on methods of purification, whether they be storage in reservoirs or filtration, that water which by comparison does not have to be treated to protect consumers against the effects of pollution, is held in greater favor.

In many American cities the water consumption is excessive, and in some it has already become a problem where to go for additional surface supplies to meet the increasing demand. Again, supplies which were once satisfactory must now be purified to protect them against chance contamination by a rapidly increasing population. All these factors tend to cause a favorable consideration of ground-water sources.

In the early days of the United States, well-water supplies were very much used, but later, when larger supplies were necessary, the

wells were forgotten. A similar condition prevailed in Germany. Herr A. Thiem* believed that the example of England, foremost in sanitary matters twenty years ago, who devoted her energies to improving and storing surface water, greatly promoted the development of surface-water supplies. These supplies were comparatively "easy to obtain." They were not dependent upon speculation, deduction or induction; they required only a small knowledge of geology or hydrology; they involved little more than pure problems of construction, and for these reasons were not difficult. The water was not hard, and very often pumping was unnecessary.

The success of the Altona filters during the cholera epidemic of 1893 inspired great confidence in filtered surface supplies, and many were introduced into European cities. They have operated efficiently and economically in most cases, and have effected a great saving of life. At the same time there has been a parallel development of the abundant ground-water supply of Northern Germany and Holland, which is a much more acceptable drinking water than a surface water on account of the greater insurance against accidental pollution due to careless filtration or to accidental mixing of highly polluted unfiltered water with the filter effluent. Several of these accidental interruptions in filter operations have occurred during the past ten years, particularly in Altona in 1886, 1887, 1888, 1891, and 1892; in Berlin in 1889 and in Hamburg quite recently. Professor Bernard Fischer* believes that this cause of epidemics is anything but rare.

At present what may be called the ground-water movement is very strong in Germany. It is favored among others by such hygienists as Flügge, Fischer, and Dunbar, and by such engineers as Thiem, Anklam, Lindley, and Grahn, and the general trend is toward the use of such supplies wherever possible. While the German hygienists make rather extravagant claims for the sanitary inviolability of such supplies and for the protection afforded to strata of water-bearing gravel by thin layers of overlying silt and clay, still, as a rule, the supplies are excellent. They are usually moderately hard, but after the removal of iron and manganese and the small amount of hydrogen sulphide which they may contain, they are of excellent appearance, cool, and safe. The faults of surface supplies, namely, changing temperature, color

* *Deutsche Vierteljahresschrift für öffentliche Gesundheitspflege*, 29, Heft 1.

odor, high first cost, and possibility of contamination, have also caused the development of many so-called gravity "spring-water" supplies. Many European cities, Vienna and Naples, for example, which are near the mountains, are supplied in this way, likewise many mountain towns and villages.

In North America few large cities have been supplied with soft ground-water, and, while Indianapolis, Memphis, and Havana have ground-water supplies which are moderately hard, up to the present time, Brooklyn, Camden, Lowell, Newton, and Pueblo (Colo.), are about the only large cities which have soft ground-water supplies. In Europe many of the largest cities, for example, Budapest, Munich, Dresden, Leipsic, Amsterdam, Brussels, etc., have moderately hard ground-water supplies; Breslau, Berlin, and Hamburg, comprising a population of more than 3 500 000, are abandoning their filtered surface-water in favor of ground-water.

One reason for the paucity of ground-water supplies is the lack of knowledge, on the part of engineers and others, of the geology and hydrology of the subterranean sources. However, great progress has been made in late years; there is more ground-water existent than was generally supposed. It is present not only in the alluvium, but in some porous rocks, such as the Potsdam sandstone. It is believed by the writer that a thorough preliminary study of water problems would reveal, in many cases, satisfactory ground-water sources. Such studies, however, should be carried on by one acquainted with hydrological methods. Often when true ground-water cannot be obtained, it may be practicable to supply consumers with artificial ground-water obtained by irrigating sandy tracts with surface-water and collecting the water from the same by suitable wells or collecting galleries.

In Germany the cost of supplying a ground-water, deferrization included, is usually less than that of a filtered surface-water. This is because higher rates of filtration may be used and also because the cost of operating the filters is less.

In his study of the Breslau supply, Herr A. Thiem compared the costs as shown on Fig. 1.

The advantages of a good ground-water over a filtered surface-water supply may be summarized as follows:

1.—Absence of color, odor, and objectionable taste.

2.—Safety.

3.—Small seasonal variation in temperature.

4.—Lower cost.

5.—Less danger from frost.

All ground-waters are not satisfactory. Those which are too hard or too alkaline and those which contain more than small amounts of chlorides or traces of iron or manganese are unfit to use in their unpurified condition. This paper does not concern itself with methods of water-softening or alkali reduction, but with the theory and prac-

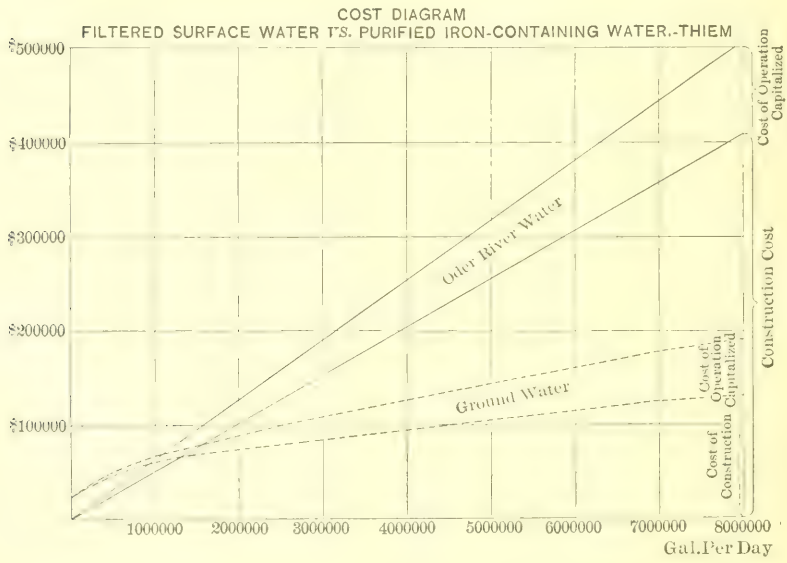


FIG. 1.

tice of the iron-removal processes in use in Europe and America. In Germany, Herr A. Thiem has coined the word *enteisenung* to describe the iron-removal process. The new Technological Dictionary will give "deferrization" as the translation of this word, which means literally "taking of iron away from" (water). Likewise one might use demanganization to describe the removal of manganese.

The deferrization and demanganization of ground-waters will be discussed under the heads: Theoretical; Historical; European Practice; Experience at Reading, Mass., and other American Water-Works; Demanganization; Design of Deferrization Plants; and Conclusion.

Theoretical.

Characteristics of Typical Iron-Containing Waters.

When typical ground-waters containing iron are freshly drawn, they are clear and colorless or nearly so, contain high amounts of carbon dioxide, sometimes considerable methane or hydrogen sulphide, and are deficient in oxygen. They have the styptic or "inky" taste which is characteristic of iron. When sufficient oxygen has had access to the water, it becomes turbid with oxidized insoluble iron, which after a shorter or longer time gathers into flocs and finally precipitates, leaving the water itself colorless and free from iron. Where there is no purification plant, this precipitation takes place in the distribution system; tea made with the water is black, rust spots appear on linen, and white plumbing fixtures are stained with iron rust. After a sudden disturbance, as after a large fire, the water is exceedingly turbid with precipitated iron oxide. In some cases a fungus, *Crenothrix,* grows in the pipes, feeding upon the organic matter and causing a precipitation of the iron contained in the water. Frequently the growth of this organism chokes service pipes and small mains, while the odors due to its death and decomposition become extremely disagreeable.

Manganese-containing waters have similar characteristics except that the precipitation is slower and the stains and deposits are much darker in color. Certain ground-waters contain a little alumina which behaves in much the same way, but, because the precipitate is white, it has never been known to cause trouble, although it is known to accompany the growth of *Crenothrix**.

Character of Ground-Water.

Like other natural products, ground-water containing iron and manganese varies greatly in its composition, as shown in Table 1.

The primary sources of iron in ground-water are the deposits of iron in the earth's crust, amounting, according to Mr. F. W. Clark,† to 5.46%, that is, the fourth in importance of the elements composing the earth. The solvent of the iron is the ground-water itself, aided by the acids dissolved in it. It has been stated by many writers, and it is quite commonly believed, that iron exists in water as a salt of one or more of the acids, chiefly as ferrous carbonate; but Whitney‡

*D. D. Jackson, *Journal,* Society of Chemical Industry, 21, 681.
†Chemist, U. S. Geological Survey.
‡"The Corrosion of Iron," *Journal,* Am. Chem. Soc., 25, 394.

TABLE 1.—Composition of Ground-Waters Containing Iron and Manganese.

(Parts per Million.)

Locality.	Worms.	Berlin, Müggelsee.	Posen, Shallow well.	Posen, Deep well.	Lowell, Mass.	Reading, Mass., Nov., 1891.	Reading, Mass., Jan., 1900.	Reading, Mass., June, 1902.	Provincetown, Mass., Average.	Marblehead, Mass., Average.	New Orleans, La., Deep well.	New Iberia, La., Deep well.
Color	.....	.....	100.	1000+	8.	4.	39.	105.	92.	7.	1400.	11.
Temperature, in degrees centigrade	9.4	10.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Iron	19.5	1.12	12.	0.6	3.1	13.3	3.9	1.4	3.6	1.	3.2	8.6
Manganese	.....	.....	1.2	0.0	.....	.....	.....	.....	.....	.....	.....	.....
Oxygen consumed	1.3	3.9	24.	246.	0.7	.....	3.3	7.4	7.1	0.4	14.3	3.0
Free ammonia	0.0	0.280	0.7	2.6	0.080	0.008	0.110	0.088	0.124	0.065	0.029	3.3
Alb. ammonia	.....	.....	.....	.....	0.031	0.270	0.064	0.132	0.183	0.022	0.181	0.09
Nitrites	0.0	0.000	0.03	0.00	0.001	0.001	0.001	0.001	0.001	0.002	0.400	0.00
Nitrates	trace	0.0	0.0	0.0	0.242	0.20	0.04	0.06	0.055	0.61	0.00	0.00
Chlorine	24.	5.6	18.	178.	2.7	4.2	5.4	4.1	2.6	29.5	.....	10.5
Total hardness	.....	166.	272.	160.	18.	116.	33.	31.	25.	75.	403.*	43.1*
Residue on evaporation	350.	242.	464.	645.	44.	264.	96.	75.	105.	174.	857.	500.
Calcium, Ca	158.	.....	88.	43.	.....	.....	.....	.....	.....	.....	.....	.....
Magnesium, Mg	.....	.....	13.	12.	.....	.....	.....	.....	.....	.....	.....	.....
Sulphuric acid, SO_4	.....	20.	47.	37.	.....	.....	.....	.....	.....	.....	.....	.....
Free carbon dioxide	.....	.....	16.	46.4	.....	.....	.....	.....	.....	.....	.....	.....
Hydrogen sulphide	Present	Present	Present	0.	.....	Present	Present	Present	Present	.....	.....	.....

* Alkalinity (Na_2CO_3). † Total carbon dioxide.

has proved that absolutely pure water, free from oxygen and carbonic acid, will dissolve an appreciable amount of metallic iron. The force which causes this solution (the "solution pressure") is an electrical one. Iron has a natural tendency to go into solution. It is a well-known fact that if a piece of bright iron be immersed in a solution of copper sulphate, the metallic copper will be plated out on the iron, and the iron will replace the copper in the solution as shown by the following equation:

$$Fe + CuSO_4 = Cu + FeSO_4.$$

The explanation of this phenomenon, according to the modern theory of solutions, is that copper sulphate is an electrolyte* and in a solution is partially dissociated into its ions, Cu and SO_4. There are present in the solution the positively charged copper ions which have less solution pressure than the positively charged iron ions; hence the former are attracted to the negatively charged metallic iron; the electric charges are neutralized, and metallic copper is precipitated. This action continues because the copper is a conductor and does not re-dissolve in the presence of iron for the reason explained above.

Water, like copper sulphate, is dissociated to a small degree into hydrogen, H+, and hydroxyl, OH—. Its formula may be written as

$$H^+ - OH^-.$$

From an electro-physical standpoint, hydrogen is a metal and acts like copper; that is, it plates out on the metallic iron at the negative pole, while the iron takes its place in solution. However, the hydrogen is not liberated readily upon the surface of the iron, and soon the condensation of gaseous hydrogen, which is a non-conductor, on the surface of the iron checks the action. In order that the solution of iron may proceed further, the force which prevents the hydrogen from being liberated as a gas on the surface of the iron must be overcome. This is accomplished by what the physical chemists term over-voltage; that is, it is necessary to increase the concentration of the hydrogen ions in the solution in order to force the liberation of hydrogen gas, or, as one may say, that the bubbles of gas may be crowded off the surface of

*An electrolyte is a solution which conducts current. The modern theory of solutions considers that electrolytes, when dissolved in water, are separated or dissociated into component parts called ions. Thus common salt is dissociated into its Na ions, which are positive, and its Cl ions, which are negative; Gypsum, $CaSO_4$, into Ca ions which are positive and SO_4 ions which are negative. Any compounds thus dissociated, that is, separated into their ions, conduct current.

the iron. Then the iron goes into solution as ferrous hydrate, hydrogen being evolved. The equation is as follows:

$$Fe + 2H\,OH = Fe\,(OH)_2 + 2H.$$

Acids like carbonic acid ($H_2\,CO_3$), or sulphuric acid ($H_2\,SO_4$), added to water greatly increase the concentration of the hydrogen ions and promote solution; while alkalies like lime Ca (OH)$_2$, or caustic soda Ma OH, increase the concentration of the hydroxyl ions and check solution.

Dr. Allerton S. Cushman,* Professor W. H. Walker, and others† have shown that a piece of ordinary iron, which is never homogeneous, immersed in water acts like an electrolytic couple, that is, it produces electric currents, the particles of metallic iron acting as positive poles, and the particles of carbon, graphite or scale, as negative poles. They have shown that the hydrogen is deposited and later given off at the negative poles, while the solution of iron in the form of ferrous hydrate takes place at the other. In an acid water, the one which corrodes iron most readily is that which has the largest excess of hydrogen ions.

The conclusion from all these studies is that, in the absence of oxygen, iron exists in water as the ferrous or unoxidized hydrate. This is quite soluble in water; in the amounts usually found in water, entirely soluble.

The iron in water may be from vegetable or mineral sources. The mineral sources are the iron silicates, carbonates, sulphates, chlorides, phosphates, oxides, etc. The chief vegetable source is the iron in plants, living and dead, beginning with the green chlorophyl of the leaves of plants, a compound of iron, and extending through leaf mould, peat, and lignite, to anthracite. Generally speaking, all these compounds are derivatives of oxides.

These deposits of iron are more or less soluble in water which is free from oxygen, and especially so in water containing acid. The degree of solution is proportional to the amount of the acid, because it is the excess of hydrogen ions which effects solution. Carbonic acid is most important in this connection, not because it is a strong acid, which it is not, but because its amount is usually far greater than that of any other acid present in water.

The primary source of carbonic acid is the organic matter in the

*Bulletin No. 30, Office of Public Roads, U. S. Dept. of Agriculture.

†"The Corrosion of Iron and Steel," *Journal*, Am. Chem. Soc., 29, 125.

soil. The rain water which falls on the earth is saturated with oxygen, and, during its passage through the ground, it will not dissolve iron from mineral although it may from vegetable sources. However, the organic matter undergoes oxidation, oxygen is absorbed and carbon dioxide is evolved—the water loses its oxygen and becomes charged with carbonic acid. More extended decomposition of organic matter in the absence of oxygen may produce hydrogen sulphide and methane. One or both of these gases may be dissolved in ground-water according to its previous history, and increases its solvent power. The occurrence of a black, peaty layer overlying white sand containing iron, the white sand in turn overylying red sand containing oxidized iron, is a familiar sight to all engineers who have made excavations. Here the carbon dioxide produced by the decomposition of the organic matter has caused the solution of the iron from the upper layer, has carried it down through the white sand, and has kept it in solution until the iron has become oxidized and re-precipitated in the layer beneath. Wells sunk beneath such soil may be free from iron at first, but continued pumping may cause so much water to pass through the soil that it is robbed of its small amount of oxygen when the iron goes into solution and appears in the well water. This cause is frequently met in practice.

It is stated by many geologists that carbon dioxide is frequently of volcanic origin, resulting from the decomposition of iron carbide by iron oxide at high temperatures. This may explain the presence of iron in the waters of deep wells far removed from any deposits of organic matter. The organic acids, for example, crenic and apocrenic acids, may also aid the solution of iron. The waters of certain Brazilian rivers which contain large amounts of organic acids are said to be very corrosive, and the bottoms of iron boats are attacked rapidly. The production of hydrogen sulphide in the presence of iron and organic matter results in the formation of ferrous sulphide. This is insoluble, but if exposed to the air it readily oxidizes to ferrous sulphate which is very soluble in water. This is usually the explanation of the presence of ferrous sulphate in the so-called "acid iron" waters.

As stated later, manganese sulphide behaves in a similar manner, with the formation of manganese sulphate and, in the absence of alkalies, of free sulphuric acid, which in turn effects a rapid solution of the iron and manganese with which the water comes in contact.

Iron (or manganese) in water may be: (1) In the unoxidized or ferrous condition, when it is soluble in water; (2) In the oxidized or ferric condition, when it is almost insoluble in water containing oxygen; (3) In the colloidal or semi-soluble form, when it may be oxidized, yet to all outward appearances in solution. Perhaps there is, in addition, an insoluble, vegetable compound of iron which is present in the colloidal form.

According to Schwers* such a vegetable compound of iron is produced by the oxidation of organic matter to organic acid by ferric hydrate. The organic acid thus formed combines with the iron to form an insoluble organic salt of iron which is exceedingly difficult to treat. This compound is formed after the water is drawn from the ground and after a portion of the iron has oxidized.

Ferruginous ground-waters are of three general classes: (1) Those which begin to precipitate the iron immediately after aeration, like many of the waters of North Germany; (2) Those which will hold the iron in solution indefinitely, even after aeration, like the dark brown water of deep wells or the acid waters from mines; (3) Those which precipitate a part of the iron within a few hours and hold the remainder in solution indefinitely, like the water at Reading, Mass.

Clear and colorless ferruginous ground-waters never contain oxygen. When both oxygen and iron are present, the water is either turbid or colored. In the latter case the iron is held in solution or suspension by vegetable matter.

The amount of iron in water varies from a slight trace to more than 2 500 parts per million. Manganese does not usually occur in large amounts, although on one occasion more than 220 parts per million were found in the Breslau supply. As a rule, however, the waters used for water supplies contain less than 10 parts of iron per million and only small amounts of manganese. The average iron content of certain ground-waters is given in Table 2, which also indicates roughly the geographical distribution of ferruginous water. It will be deduced that the ground-waters of Bavaria, Baden, Austria, Switzerland, Scandinavia, Italy, and parts of North America, seldom contain iron, and it is believed that the same is true of large parts of Russia, Siberia, Asia Minor, France, and the Spanish Peninsula, although analytical data are wanting in this connection. The writer has not been able

*Révue d'Hygiène, 30, 192.

to obtain very much information concerning the iron content of the waters of the British Isles. It is a well-known fact, however, that some of the hard well-waters of England contain more or less iron, and that it can be removed by the softening process.

Generally the waters which contain manganese contain still more iron. In one case, however, at Halle, the amount of manganese exceeds that of iron.

TABLE 2.—Amounts of Iron in Various Ground-Waters.

(Parts of Fe per Million.)

Locality.	Fe.	Locality.	Fe.
Aguas de Bar, Spain	61.	Indianapolis, Indiana	41.
Arad, Hungary	25.	Joplin, Mo.	0.3
Asbury Park, N. J.	9.1	Kiel, Germany	4 to 7
Aurich (Friesland)	19.2	La Grange, Mo.	1 5
Berlin, Müggelsee	1.3	La Reid, Belgium	0.97
Berlin, Tegel	1.0	Leipsic, Germany	3 to 14
Blanchimont, Belgium	21.3	Lowell, Mass.	3.1
Bloomfield, Conn.	15.	Manchester, Mass.	0.07
Brunswick, Germany	6.0	Manheim, Germany	0.13
Brée, Belgium	6.8	Marblehead, Mass.	3.87
Breslau, Germany	6 to 400.	München-Gladbach, Germany	2.3
Centreville, Iowa	3.2	Posen, Prussian Poland	7.1
Charlottenburg, Germany	1.4	Provincetown, Mass.	3.6
Chaudfontaine, Belgium	6.3	Reading, Mass.	1.21
Chevron, Belgium	40.6	Richmond, Mo.	12.0+
Cohasset, Mass.	1.3	Seraing, Belgium	0.25
Court St. Etienne, Belgium	6.3	Spa (Poulion Pierre le Grand), Belgium	36.2
Diepenbeck, Belgium	4.	Stargard, Germany	2.0
Far Rockaway, N. Y.	2.48	Superior, Wis.	1.7
Garrettsville, Ohio*	2.0	Tampa, Fla. (Sulphur spring water)	1.0
Glogau (Germany)	5.8	Tougres (Montagne de fer)	62.2
Gottelheim (Westphalia)	110.	Turnhout	12 to 20
Halle, Germany	0.8	Webb City, Mo.	213.1
Hamburg, Germany	2.0	Winona, Minn.	0.6
Hanover, Germany	0.5		
Huy, Belgium	8.		

Professor Edward Bartow, Director of the Illinois State Water Survey,† gives the variation in the amounts of iron in the waters of Illinois as follows:

In springs, from traces to 997 parts of iron per million;

In drift wells, from traces in several wells to 11 parts per million in a well at Paris, Edgar County, the higher quantities being found in the deeper wells;

In rock wells, from traces in many wells to 2506 parts per million in a deep well at Kell, Marion County.

The majority of these wells have less than 3 parts of iron per million.

*Furnished by Mr. Winthrop Pratt.

†Illinois State Water Survey, Bulletin No. 4.

TABLE 3.—Amounts of Manganese in Various Ground-Waters.

(Parts of Mn per Million.)

Locality.	Mn.	Locality.	Mn.
Arad, Hungary	Present	Halle, Germany	1.50
Babylon, N. Y.*	0.07	Hamburg, Hofbrünnen	0.45
Bayshore, N. Y.*	0.37	Hanover, Germany	Present
Berlin, Germany	Present	Patchogue, N. Y.*	0.20
Björnstorp, Sweden	3.4 to 53.4	Reading, Mass.	0.004 to 0.56
Brunswick, Germany	Present	Stargard, Germany	Present
Breslau, Germany	Trace to 110	Stettin, Germany	5.22
Calverton, N. Y.*	0.30	Superior, Wis.†	0.12

The amount of iron which will cause trouble varies with the water. Allen Hazen, M. Am. Soc. C. E.,‡ states that a water containing 0.3 part of iron per million rarely precipitates, and that 0.5 part is the allowable limit. This is a good general rule. However, certain waters in the Rhine provinces, which contain as little as 0.1 part of iron per million, deposit iron oxide in the distribution system, while some American deep well-waters which contain nearly 1.0 part of iron per million are used without complaint.

The Deterioration of Ground-Water Supplies.

Many ground-water supplies which when first put into service furnish excellent water, are found in later years to contain gradually increasing and objectionable amounts of iron. The Massachusetts State Board of Health has made a study of the 78 ground-water supplies in the State in use at the end of 1905.§ One of these supplies, Bradford, was so bad that it was abandoned; the supply at Reading has to be purified; that at Marblehead is to be purified, and many of the others now contain objectionable amounts of iron which should be removed. Table 4, abstracted from the report, shows how the amounts of iron have increased in ten years. No results of tests for manganese have been published.

Fig. 3 shows the increase in the amount of iron in the Reading water supply.

*Data furnished by A. D. Flinn, M. Am. Soc. C. E.
†Sample furnished by Mr. W. Lonnsbury.
‡"Filtration of Public Water Supplies," p. 186.
§Report for 1905.

TABLE 4.—The Amounts of Iron in Certain Massachusetts Ground-Waters.

City or Town.	To 1895.	1896-1900.	1901-1905.
Amesbury	....	0.40	0.67
Billerica	....	0.18	0.36
Brookline	0.02	0.03	0.15
Fairhaven	0.01	0.12	0.15
Framingham	0.14	0.10	0.14
Hyde Park, wells near Neponset River	0.14	0.16	0.41
Lowell, Boulevard wells	....	0.32	0.32
Marblehead	0.38	1.19	2.03
Methuen	0.13	0.09	0.28
Middleboro	0.19	0.35	0.96
Newton	0.12	0.06	0.13
Waltham	0.04	0.15	0.31
Wellesley	....	0.05	0.08
Winchendon	....	0.09	0.28
Woburn	0.01	0.02	0.05

The Board reports that:

"The deterioration is usually most marked in those sources which are located near a pond or stream, the water being derived in such cases in part from the rain which falls upon and sinks into the ground in the neighborhood of the collecting works, and in part by percolation through the ground from the neighboring surface source * * *."

"Iron is frequently present in water which has passed through peat and other soil containing a large quantity of organic matter, and which has not been subsequently purified in its passage through sand or gravel to the collecting works; and in some cases the deterioration of ground-waters has been due to the reduction in efficiency of the filtration of water which has taken up organic matter and iron from peat or muck near the surface of the ground. An example of such deterioration is found in Billerica, where the wells are located in the midst of an extensive meadow which contains at the surface a deep deposit of organic matter."

The Board also demonstrates that increased draft is often responsible for the deterioration in the character of the water. "At Lowell a great improvement was made in the quality of the water drawn from the ground in the vicinity of the Merrimack River by an extension of the collecting works." A like improvement has been observed in other places, but it is usually of a temporary character only.

There has been an unfortunate tendency in cases where the iron has increased, as in the above, to abandon the ground-water supply, with its invested capital, and seek another source. Such a ground-water, purified, would be safer than a filtered surface-water, and in most cases more economical.

Reading, Mass., Water Supply.

There is no iron-containing water in the writer's experience which has been so instructive as that from the Reading, Mass., filter gallery. Fig. 2 shows the location and construction of this gallery.

The Reading filter gallery was built in 1890 by the late M. M. Tidd, M. Am. Soc. C. E. It consists of a permeable masonry tunnel 2.5 ft. wide, 4.0 ft. high, and 240 ft. long, placed in the water-bearing gravel beneath the littoral marshes of the Ipswich River which frequently overflow its banks. The bottom of the gallery is 22 ft. below the surface of the ground, while the normal ground-water level, even during droughts, is rarely 10 ft. below the surface. Water is pumped

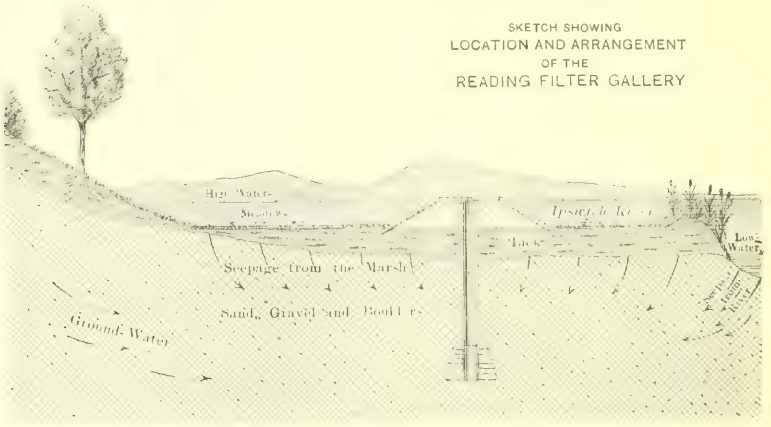


FIG. 2.

from the gallery for about 6 hours per day, on an average, and, at certain times of the year, the ground-water level may be depressed more than 20 ft. below the surface just before pumping is finished. The water in this gallery is a mixture of three kinds, namely,

1. The true ground-water which is flowing from the higher land toward the river;

2. The seepage from the peaty matter immediately above the gallery; and

3. The water which filters directly into the gallery from the river.

All these components are different in character, and, consequently, the mixture not only varies with the season, but, at times, with the hours in a single day. According to Desmond FitzGerald, Past-Presi-

dent, Am. Soc. C. E., more than half the water collected in 1893 came from the Ipswich River. If the water from the ground itself could be kept separate, it probably could be purified by simple aeration and filtration. The river water is somewhat colored with vegetable matter, contains oxygen, and could be coagulated with sulphate of alumina without difficulty. The seepage from the overlying peaty layer is rich in humus-like bodies, and is the cause of most of the difficulties which have been experienced. When, after a prolonged drought, there comes a heavy rain or a sudden overflowing of the meadow, the humus substances are dissolved to a considerable degree, and, seeping downward, become mixed with the gallery water. These substances form compounds with iron which without special treatment will not coagulate and precipitate for weeks, although the iron is oxidized quite readily. When the water containing humus matter and iron is heated, the color deepens. Boiling, which usually hastens the precipitation of iron from a ground-water, only serves to form more of this compound of iron with organic matter and to keep the iron longer in suspension. Carbon dioxide, however, prevents the formation of this compound, and water fresh from the wells and treated with carbon dioxide before the compound is formed can be freed from iron more readily than is the untreated water. On the other hand, the addition of carbon dioxide after the compound has once formed seems to have little or no effect upon the precipitation of the iron.

These facts are shown by the following experiments:

Experiment No. 1.—Effect of Boiling.—A sample of water was collected on February 17th from the Reading filter gallery and boiled 5 min. During boiling the color increased from 70 to 120. The amount of iron in the sample was 3.4 parts per million, 3.3 parts of which were in the oxidized condition. The boiled sample was cooled, aerated, and allowed to stand for 24 hours. It was then filtered through paper and found to contain 2.9 parts of iron per million. The filtered untreated sample contained 1.6 parts of iron per million.

Experiment No. 2.—Effect of Addition of Carbon Dioxide to Fresh Sample.—To two samples of fresh Reading water containing 3.4 parts of iron per million were added: (*a*) 50 cu. cm. of distilled water charged with carbon dioxide; (*b*) 50 cu. cm. of plain distilled water. The latter sample was the control.

After two weeks in tightly sealed bottles, the following amounts

of iron were found in the filtered samples: (*a*) 0.3 part per million; (*b*) 3.3 parts per million. After a month, the filtered untreated water still contained iron.

Experiment No. 3.—Effect of Addition of Carbon Dioxide to Sample of Stale Water.—A sample of water, not freshly drawn, containing 2.5 parts of iron per million, was treated with carbon dioxide as in Experiment No. 2. After three days the treated sample filtered through paper contained 1.4 part, and the untreated or control sample, 1.1 part of iron per million. After one week the amount of iron in the treated sample had dropped to 1.0 part, and in the control to 0.92 part per million.

Analytical Characteristics of Typical Reading Waters.

Table 5 gives the results of three analyses of samples collected on February 17th, April 2d, and July 4th, 1908, respectively.

The first sample represents water which contains the iron and organic matter compound which is so difficult to treat. The second sample, on the contrary, is one from which the iron is removed with less difficulty. The third sample contains the minimum of seepage water. The most important hindrance seems to be the organic matter, as shown by the color and albuminoid ammonia, while the presence of nitrite is an indication of seepage from the overlying peaty deposits.

TABLE 5.—Results of Analyses of Reading Water.

(Parts per Million.)

	Reading, Mass., Filter Gallery.		
Source of sample			
Date of collection	Feb. 17, 1908	Apr. 3, 1908	July 4, 1908
Date of examination	Feb. 18, 1908	Apr. 4, 1908	July 6, 1908
Temperature, in degrees centigrade	6.5	.....	12.
Odor: Cold: 20° cent	1–Vegetable	0–	0–
Odor: Hot: 90° cent	2– "	1–Vegetable.	1–Vegetable.
Turbidity: Silica standard	.....	2	.....
Color: Platinum standard	70	40	53
Color of boiled water	120	74	85
Increase in color	50	34	32
Oxygen consumed	6.9	4.5	.....
Nitrogen as: Free ammonia	0.130	0.068	0.054
Nitrogen as: Albuminoid ammonia	0.178	0.092	0.102
Nitrogen as: Nitrites	0.006	0.002	0.000
Nitrogen as: Nitrates	0.15	0.20	0.10
Chlorine	10.	7.5	.....
Hardness, by soap method	50.	.....	23.
Alkalinity	10.	9.	12.
Residue on evaporation	130.	79.	102.
Iron	3.3	1.8	1.91
Iron in filtered sample	3.3	1.6	0.90
Carbon dioxide (free)	.....	28.	36.
Dissolved oxygen	1.4	.....	1.55
Manganese	0.33	0.004	0.000

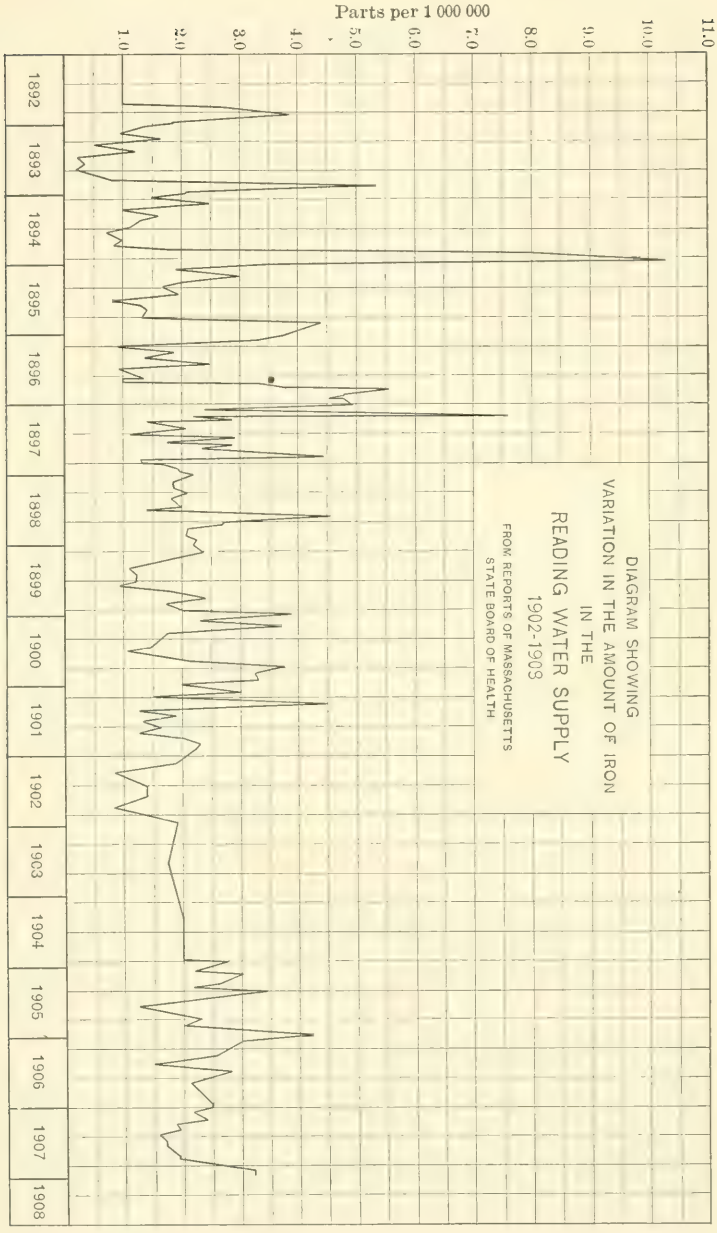

Parts per 1 000 000
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
1892
1893
1894
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
1905
1906
1907
1908
DIAGRAM SHOWING
VARIATION IN THE AMOUNT OF IRON
IN THE
READING WATER SUPPLY
1902-1903
FROM REPORTS OF MASSACHUSETTS
STATE BOARD OF HEALTH

FIG. 3.

In a single day some of the constitutents may vary as shown by tests made on July 4th, 1908, the results of which are given in Table 6.

TABLE 6.

Time.	Iron.	Parts per Million.	
		Carbon dioxide.	Dissolved oxygen.
7.15 A. M.	4.30	35.	2.15
8.45 "	2.28	41.	0.80
9.00 "	....	38.	....
9.30 "	2.20	37.	0.46
10.00 "	....	35.	2.15
11.00 "	1.45	36.	0.80
12.00 M.	1.70	39.	0.86
1.00 P. M.	....	....	2.39
1.15 "	1.70	....	2.42
1.30 "	....	....	1.23
2.00 "	2.20	37.	2.98
3.00 "	....	34.	1.41
3.40 "	1.90	....	1.89

Iron accompanied by color and organic matter coagulates very slowly. An average sample of water collected on February 20th, 1908, was allowed to stand in a bottle exposed to the air. At the beginning of the experiment it contained 1.5 part of dissolved iron per million. After various periods of time it contained the following amounts:

Elapsed time.	Dissolved iron, in parts per million.
Beginning	1.5
1 month	0.8
1.5 months	0.7
2.0 "	0.65
2.5 "	0.6

The quantity of iron in a ground-water may vary greatly, as shown on Fig. 3. This diagram gives the amount of iron in the Reading water since 1892, as determined by the Massachusetts State Board of Health.

Naturally, the methods for purifying such a water as that from Reading must be varied because the water varies in character. The iron exists in many physical states, and its removal is at times beset with many hindrances. Sometimes the chemical reactions (simple oxidation) are not interfered with; at other times the iron may be oxidized, but in such a physical state that it will not be precipitated

within a reasonable time. It may be said to be in the colloidal condition—or in semi-solution.

Properties of Colloidal Mixtures.

The theory of the iron-removal process has a physical as well as a chemical basis, and it is necessary at this point to define briefly the nature of solutions in general in order that the nature of the dilute solutions of iron in natural water in particular may be better understood. Any brief résumé must be inadequate, but more detailed information may be found in recent chemical literature.*

For the purposes of discussion, all solutions or suspensions in water are called mixtures, and of these mixtures physical chemists recognize three groups:

A.—Solutions of crystalloids (*e. g.*, salt in water).

B.—Colloidal solutions (*e. g.*, gelatine in water).

C.—Colloidal suspensions (*e. g.*, extremely finely divided clay in water).

Nearly fifty years ago the English scientist, Thomas Graham,† separated all substances which form solutions, that is, homogeneous mixtures with water, into two groups, namely, crystalloids and colloids. Common salt is a crystalloid. It dissolves in water and can be crystallized from water unchanged. It exerts osmotic pressure, that is, it will diffuse through parchment. The solution is not viscous. Gelatine is a colloid. It dissolves in water to form a viscous solution, and will solidify or gelatinize when the solution is cooled. It will not dialize‡ through parchment, and, therefore, the osmotic pressure is very low.

The names arose because the crystalloids usually separated from their mixtures as crystals, while the colloids separated in an amorphous or glue-like state, hence "colloid" from the Greek word, *"kolla,"* meaning glue.

*A. Lottermoser, "Anorganische Kolloide"; A. Müller, "Die Theorie der Kolloide"; "Allgemeine Chemie der Kolloide"; G. Bredig, "Anorganische Fermente"; A. A. Noyes, "The Preparation and Properties of Colloidal Mixtures"; *Journal*, Am. Chem. Soc., 27, 850; "Zur Kenntniss der Kolloide."

†*Annales de Chimie et de Physique*, 1-77, 121 (1862); 3, 127 (1864).

‡Dialize, from the Greek *"dia"* (through) and *"luein"* (to loose), meaning to separate by taking advantage of the unequal diffusion of certain substances especially through permeable membranes; for example, if a parchment bag filled with salt solution be suspended in running water, it will be observed that the salt diffuses through the parchment bag after a short time, while a solution of gelatine similarly treated would remain within the bag indefinitely. This method is used to separate colloids from crystalloids.

Crystalloids diffuse through colloids, but colloids are impenetrable to other colloids; for example, if a stick of agar (a jelly made from a Japanese seaweed) be immersed in a solution of salt, it will be found that the salt will diffuse through the agar with almost the same rapidity as through water, while, if the agar be immersed in melted gelatine, no penetration will occur.

Both crystalloids and colloids in solution pass through filter paper unchanged, but the latter do not penetrate parchment. Of these colloidal mixtures there are two classes. Glue is typical of one class. When its mixture with water is evaporated, it gelatinizes and can be recovered and re-dissolved repeatedly. The addition of a moderate amount of a substance like salt does not cause it to coagulate and precipitate. The mixture is more viscous than water, and the viscosity increases with the amount of glue dissolved.

The second class of colloidal mixtures is entirely different. It is possible to divide metallic gold into such fine particles that it will remain suspended in pure water indefinitely, and, furthermore, will offer no more resistance to the passage of light than that afforded by solutions of certain crystalloids. This mixture, ruby red in color, will pass through filter paper unchanged, but, unlike glue, cannot be re-dissolved in water after being precipitated, without long-continued shaking and agitation of the particles. By means of the ultramicroscope and special methods of lighting, particles of gold as small as 0.000002 mm. have been measured, and it is stated by some that the particles in this mixture approach the gold molecule in size. The mixture is permanent if the water be pure, but if a small quantity of any salt (any electrolyte) be added to the mixture, the gold is immediately precipitated and cannot be resuspended except by prolonged shaking with pure water. This mixture is not viscous as are those of the first class. Many other substances act like metallic gold, for example, filter paper, kaolin, metallic sulphides, and even glass. Professor A. A. Noyes calls the first class (viscous, gelatinizing colloidal mixtures not coagulated by salts) colloidal solutions, and the second class (non-viscous, non-gelatinizing, but readily coagulable mixtures) colloidal suspensions.

The dividing lines between these groups have not as yet been clearly defined. In fact, there is some doubt if they can be defined. To illustrate, it has been shown by experiment that after a homogeneous

solution of ordinary soda and water is whirled rapidly in a centrifugal machine for a considerable time, the concentration of the soda in the solution is greatest in the circumference and least at the center. This leads one to ask if the differences between these various bodies are not simply differences in the sizes of the particles, differences in degree rather than in kind. Is not this true of colloids at least, which, although they pass through a coarse filter like paper, cannot pass through a finer filter like parchment, perhaps because of the well-known greater sizes of their molecules? Are not colloidal suspensions simply mixtures where the substances in suspension have been subdivided into particles so fine that the effect of gravity is overweighed? No satisfactory answer to this question has yet been given, but without going deeper into the chemical theory, the writer would state that electrical forces have something to do with keeping some of the bodies in solution or suspension, that is, the particles are kept in constant motion by electric force and cannot subside.

Colloids suspended in water may be separated into two groups: First, those which when subjected to the influence of an electric current migrate toward the negative pole, that is, move with the current. These are called positive colloids. Second, those which migrate toward the positive pole, that is, against the current. These are called negative colloids.

The problems connected with the purification of ground-waters containing iron and manganese are those of the coagulation and precipitation of the positive suspended colloids, such as the hydrates of the metals (iron hydrate, manganese hydrate, etc.). The coloring matters in some of the highly colored deep-well waters, like the deep-well waters of the Mississippi Delta and those proposed for the water supply of Posen (Prussian Poland), and, perhaps, the Dismal Swamp, are largely negative colloids. The finely divided suspended clay in muddy river water is also a negative colloid.

Fig. 4 illustrates the classification of colloids.

Theory of the Deferrization Process.

Iron (or manganese) in the water of springs and wells is in the form of a colloidal mixture. If the water remained pure, this mixture would be permanent. Soon after aeration, however, the salts dissolved in the water, the presence of suspended matter in contact with filter-

ing material, one or all, cause the colloid to coagulate and to precipitate as amorphous matter; in the case of iron, as amorphous ferric hydrate which when dried becomes ordinary iron rust—ferric oxide. The chief difficulty in the deferrization process is to precipitate the colloidal iron hydrate within a reasonable length of time, and all schemes and devices try to accomplish this end.

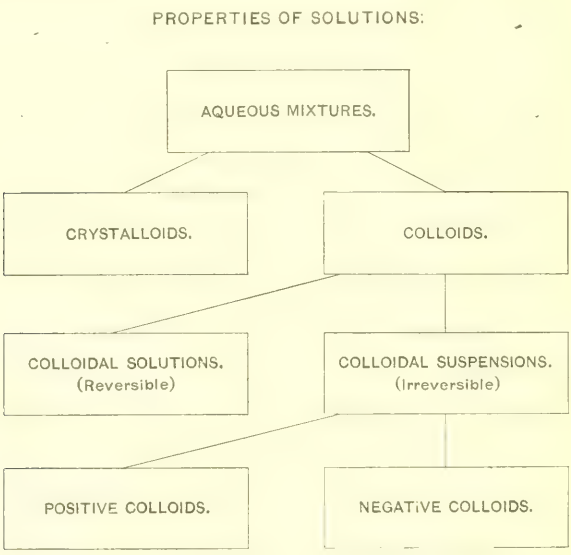


FIG. 4.

Herr A. Schmidt and Herr K. Bunte, of the Technological High School at Karlsruhe*, and others, have made studies of the chemical reactions which take place during the deferrization process. The process may be divided into phases as follows:

1.—The first requirement for the precipitation of iron is the presence of oxygen by which the oxidation of the iron is effected.

2.—The oxidation of ferrous hydrate depends on hydrolysis or the decomposition of water, as expressed in the following equation:

Ferrous Hydrate. Water. Oxygen. Ferric Hydrate. Water.

$$2Fe\begin{matrix}OH\\OH\end{matrix} + 2H\,OH + O = Fe\begin{matrix}OH\\OH\\OH\end{matrix} + H\text{-}OH$$

3.—The ferrous hydrate becomes oxidized into the insoluble ferric

*Schillings Journal für Gasbeleuchtung und Wasserversorgung, loc. cit.

hydrate which exists in the form of a colloidal suspension and remains in solution until finally

4.—The transformation of the colloidal suspension into the insoluble, gelatinous ferric hydrate is effected by various conditions which exert a helpful influence on this action, at last causing the iron to precipitate as amorphous ferric hydrate.

The discussion of the subject may be separated into divisions corresponding to the different steps in the process, namely: The absorption of oxygen; the oxidation of the iron; and the precipitation of the oxidized iron.

The Absorption of Oxygen.—When it is realized that 1 part of oxygen will oxidize 7 parts of iron, $2\ FeO + O = Fe_2O_3$, that is, 1 cu. cm. of oxygen will oxidize 10 mg. of iron, and that water under ordinary barometric pressure (760 mm.) and ordinary temperature (20° cent. or 68° fahr.) will absorb a maximum of 9.19 parts of oxygen per million, or 6.32 cu. cm. per liter, it will be realized how easy it is to supply the necessary oxygen to a water to be purified. Often a leaky suction, badly packed glands, or a small air-cock will admit a sufficient quantity. Often the filtering layer in pressure filters filled with shavings, bone charcoal or magnetite will store up enough oxygen during washing or when standing idle to oxidize the iron in the water filtered between times. If this were not so it would be as impossible to explain the success of closed filters under certain conditions as it is easy to explain why these filters fail when there is more iron present than oxygen to oxidize it.

Neither oxides of tin nor the iron ores which have been used as filtering material give up their combined oxygen to water, but Schmidt and Bunte* found that 575 g. of brown hematite contained 70 cu. cm. of various gases which had condensed on the surface of the material, while Dunbar and Kryck* have shown how oxygen is absorbed from water and air by the moist ferric hydrate which collects on the surface of the gravel and sand in aerator or filter while the devices are standing empty.

This explains why aeration is not always necessary with intermittent filters, and why Piefke* found that his coke aerator required three months to reach its maximum efficiency, that is, three months to collect a good coating of moist iron hydrate.

Loc. cit.

Water standing in tanks and reservoirs absorbs oxygen very slowly. In 6 hours, under ordinary conditions, it rarely penetrates to a depth of more than 4 in. The air and water must be brought into more intimate contact by shaking, spraying, or by cascades; by letting the water fall through towers filled with either coke, brick, broken stones, wooden slats, or by any other arrangement which will expose the water in thin layers to the air.

Oesten* has observed the increases shown in Table 7, in dissolved oxygen when a water was sprayed from varying heights.

TABLE 7.

Height of Fall.		Oxygen, absorbed, in parts per million.
Centimeters.	Inches.	
10	4	1.21
25	10	1.79
50	20	2.52
100	40	6.50
200	80	7.33

In an aerator tower, 3 ft. high, filled with 1-in. broken stone, at Reading, Mass., the writer observed the following degree of absorption of oxygen, the period of aeration being about 5 sec.

	Dissolved oxygen, in parts per million.
Well water	2.0
Aerated water	3.20
Absorbed in 5 sec.	1.20

Thus it will be seen that it is a very easy matter to supply water with the necessary oxygen.

Dunbar and Kryck have shown that iron can be oxidized when the minimum amount of oxygen necessary to combine with the iron is present; and, furthermore, have proved that the desirability of removing the carbon dioxide has been over-emphasized. To be sure, other things being equal, the presence of carbon dioxide retards the oxidation of iron, and, furthermore, Dunbar and Kryck worked with a water which contained many of those salts which assist precipitation and, therefore, might minimize the effect of the carbon dioxide.

*Schillings Journal für Gasbeleuchtung und Wasserversorgung, 34, 283.

On one occasion, however, they added 297 parts of carbon dioxide per million to an aerated water which already contained 150 parts of carbon dioxide and succeeded in reducing the iron to 0.2 part per million by subsequent filtration, which result was neither better nor worse than those obtained without this excessive amount of carbon dioxide. The filtered water contained 447 parts of carbon dioxide per million.

Not only is excessive aeration unnecessary in most cases, but in some it is exceedingly undesirable, because it renders the precipitation of the iron more difficult. It has already been shown that compounds of iron and humus matter are difficult to treat, that they result usually from a mixture of the seepage from peaty deposits with iron-containing ground-water, and that the presence of carbon dioxide which would be removed by excessive aeration prevents the formation of this compound and allows the iron to be oxidized and removed.

One advantage in removing carbon dioxide by aeration is the diminution of subsequent corrosive action in the distribution system.

Oesten* observed at the Zoological Garden, in Berlin, that iron was dissolved from the service pipes by the aid of the carbonic acid left in the water after single filtration, as shown by Table 8, which gives analyses of samples from different parts of the Garden.

TABLE 8.

	Dec. 10th, 1907. Pumps working. FeO.	Dec. 17th, 1907. Pumps idle. FeO.
1. Raw water	2.04	3.77
2. Filtered water	0.26	0.30
3. Old bird house	0.88	0.51
4, Old ape house	0.13	0.17
5. Deer yard	0.26	3.66
6. Pheasant house	0.35	0.30
7. Hippopotamus house	1.41	5.15
8. Elephant house	....	1.07

These results were confirmed by another set of analyses of samples taken on January 11th, 1908, when the filtered water was found to contain 0.21 part, the double filtered water less than 0.01 part, and the water from the old ape house, 0.13 part of ferrous oxide per million.

The solvent effect of the water has been greatly reduced by double filtration with the consequent aeration.

*Schillings Journal für Gasbeleuchtung und Wasserversorgung, 51, 203.

Oxidation of the Iron.—The oxidation of iron is effected by hydrolysis, that is, the molecule of water, H OH, is split into H and OH, its ions; the OH ion combines with the ferrous hydrate to form the ferric hydrate, while the hydrogen unites with the free oxygen to form water. This action (hydrolysis) is retarded by acids and accelerated by alkalies and by contact with rough surfaces such as the material used in filters and aerators, or the ferric hydrate which has already been precipitated. It is what is known as a "time reaction," and may be expressed mathematically by Prinz's* formula, as follows:

A = original amount of ferrous (unoxidized) iron,

t = time since oxygen was admitted—elapsed time,

x = iron oxidized during elapsed time.

$$K = \frac{1}{t} \times \frac{A}{A - x} = \text{Coefficient.}$$

K is a coefficient which varies directly with speed of reaction, and waters showing high values of K are most readily purified. Many workers, including Proskauer,† Piefke,‡ Fischer,§ Luebbert,|| Dunbar,¶ Darapsky,** Schmidt and Bunte,†† and Clark,‡‡ have discussed this reaction from many points of view, and the subject will be dismissed with two illustrations:

1.—The influence of an acid, for example, carbonic acid.

The experiment by Schmidt and Bunte, in Table 9, with two waters containing 9.5 parts of iron per million and differing in their carbon dioxide content only, shows how the presence of carbon dioxide retards the oxidation of iron.

The value of K in the water rich in carbon dioxide averages 1.8; in the other water, 15. This means that a fourfold increase in the amount of carbon dioxide has decreased the speed of oxidation eightfold. G. Just§§ has stated that in solutions of ferrous carbonate the velocity of reaction is inversely proportional to the square of the concentration of the carbonic acid.

**Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 35, 164.
†Proskauer, *Zeitschrift für Hygiene* (1890), 9, 148.
‡Piefke, *loc. cit.*
§Fischer, *Zeitschrift für Hygiene* (1893), 13, 251.
||Luebbert, *Zeitschrift für Hygiene* (1895), 20, 397.
¶Dunbar, *Zeitschrift für Hygiene* (1896), 22, 68.
**Darapsky, *Gesundheits-Ingenieur*, 1904.
††Schmidt and Bunte, *loc. cit.*
‡‡Clark, Report, Mass. State Board of Health, 1891; also, *Journal*, New England Water Works Assoc., 11, 277.
§§Berichte der deutschen chemischen Gesellschaft, 40, 3695 (1907).

TABLE 9.—The Influence of Carbon Dioxide on the Speed of Precipitation of Iron from Water.

A WATER RICH IN CARBON DIOXIDE.

Elapsed time since the beginning of reaction, in hours.	Iron found in the solution, in parts per million.	Free CO_2, in parts per million.	Oxygen, in Cubic centimeters per liter.	K
0	0.5	270	4.07	1.3
4.25	9.0	....	5.35	1.6
8.25	8.3	192	4.50	1.5
24	6.7	150	4.55	2.3
33	4.45	145	4.86	2.2
48	3.3	140	4.90	2.1
96	1.26	126	5.16	2.0
120	0.82	117	5.20	1.7
168	0.51	92	5.35	...

A WATER LOW IN CARBON DIOXIDE.

0	9.5	60	4.68	25
3	4.5	58	5.08	16
7	3.37	55	5.08	13
24	0.41	38	5.2	12
31	0.26	38	5.35	9
48	0.12	38	5.4	..

2.—Influence of surface contact.

According to the Henry-Dalton law, oxygen will displace carbon dioxide in water, that is, oxygen will "wash it out" of water if opportunity be given. This exchange of gases is promoted by contact with rough surfaces.*

Faraday has shown that oxygen and other gases are condensed on the surfaces of solid bodies.

Dunbar and Kryck† have given examples of how an intermittent filter effects an exchange of gases. During their experiments with intermittent filters, it was observed that directly after an idle period the amount of dissolved oxygen in the filter effluent was more and the amount of carbon dioxide less than in the influent, but toward the end of the period of operation the oxygen almost disappeared from the effluent, while the carbon dioxide increased in amount. The explanation is that the oxygen is condensed on the surfaces of the filter material during the idle period. Then the filter is put into service, and the water seizes upon the stored oxygen until the supply is exhausted; meanwhile the filter material absorbs carbon dioxide from the

*Piefke, *loc. cit.*

†*Loc. cit.*

water until it at last becomes saturated. At such times the effluent often contains more carbon dioxide than the influent. A short period of rest suffices for the oxygen of the air to displace the carbon dioxide absorbed by the filter during operation.

Schmidt and Bunte* have also shown that the removal of carbon dioxide is aided by a multiplicity of contact surfaces. They filtered a water, after various periods of standing in contact with air, through filter paper and new, pure, quartz sand, respectively. Before treatment it contained 9.5 parts of iron and 307 parts of free carbon dioxide per million and 4.13 cu. cm. of dissolved oxygen per liter.

TABLE 10.—Influence of the Presence of Sand on the Speed of Precipitation of Iron From Water.

Elapsed time since the beginning of reaction, in hours	Iron found in the solution, in parts per million.	Free CO_2, in parts per million.	Oxygen, in cubic centimeters per liter.	K
Without sand:				
0	9.5	307	4.13	22
0.25	9.0	304	4.87	15
0.75	8.5	278	5.2	12
1.25	8.2	184	5.24	10
1.75	8.0	146	5.4	13
2.25	7.5	115	5.61	13
3	6.5	75	5.65	17
5.14	3.8	45	5.93	22
6.5	2.2	39	6.05	..
With sand:				
0	9.5	307	4.13	74
0.25	7.9	181	4.76	39
0.75	7.1	101	5.08	30
1.25	6.5	70	5.2	25
2.25	4.7	47	5.4	42
4.75	1.32	27	5.94	68
6	0.16	12	6.0	..

The effect of contact with sand on the speed of reaction is very noticeable. The value of K when sand was present was about three times as great as when it was absent.

Iron exists in water in company with acids other than carbonic, for example, hydrochloric, nitric, sulphuric, etc. The expression "in company with" rather than "combined with" is more in accordance with modern chemistry and engineering, and is used purposely. Were

Loc. cit.

a ferruginous water a concentrated solution, a different idea might be entertained, but where the dilution is so very great, the greater part of the iron is in the form of the hydrate, positive ions accompanied by acids—negative ions. Therefore, ground-waters are solutions of iron and acids, rather than solutions of iron salts. Waters containing hydrochloric and sulphuric acids and little alkali are difficult to purify because the presence of acid prevents hydrolysis, and thus checks the reaction.* Unlike carbon dioxide, these acids remain in solution and are not removed by aeration. The iron cannot be completely removed from waters containing free acid without the addition of an alkali.

The converse is also true, namely, the presence of an alkali, especially a caustic alkali, greatly favors the reaction, because it introduces hydroxyl (OH) ions, the excess of which hastens the action between the ferrous hydrate and the hydroxyl ions already present in the water.

It should be borne in mind that most natural waters are alkaline, and that the presence of free mineral acid is largely of theoretical interest.

The Precipitation of Oxidized Iron.—When iron oxidizes and precipitates it becomes iron hydrate, that is, iron-rust plus water. The particles may be so fine, however, that they remain in colloidal suspension indefinitely; in fact, this colloidal suspension is one of the transition stages between dissolved iron and precipitated iron-oxide sludge. The passing out of the colloidal stage is called coagulation. As coagulation begins, the fine particles attract and adhere to one another to form aggregates. These aggregates become large enough to be precipitated by gravity, and subside. Meanwhile they attract other aggregates until the liquid is filled with visible flocs of slowly subsiding ferric hydrate. This process is not instantaneous; it may be finished in a few minutes or it may require months, according to the character of the water under consideration; and the aim of engineering practice is to reduce the time to a minimum.

Factors Which Influence Coagulation.

W. Spring† has expressed the opinion that the electrical forces which hold iron in solution must be neutralized to cause its precipita-

*See p. 1346.

†*Recueil des Travaux Chimiques des Pays-Bas et de la Belgique*, 19, 204-230 (1900).

tion. Without elaborating this statement it may be said that among the conditions which influence the coagulation and precipitation of iron hydrate are the following:

Presence of electrolytes;
Presence of alkalis;
Agitation and friction;
Contact with filtering material or precipitated iron hydrate;
Temperature;
Presence of humus compounds;
The relative concentrations of positive and negative suspended colloids, respectively.

Presence of Electrolytes.—If a weak solution of common salt be added to a colloidal suspension of iron hydrate in pure water, the hydrate coagulates in a brief time. This is because electrolytes precipitate colloids from colloidal suspensions. To understand this phenomenon it is necessary to consider that salts do not exist as such in aqueous solutions, but they are dissociated more or less completely into their component parts which are called ions. The force which causes this dissociation is an electrical one, the reverse of the well-known technical electrical process for transforming common salt into its constituents, namely, sodium (hydrate) and chlorine (gas), where the sodium ion in the solution migrates with the current to the negative pole; the chlorine ion against the current to the positive pole.

Likewise, calcium carbonate ($Ca\ Co_3$) in solution partially dissociates into calcium (Ca) and carbonic acid (CO_3), and magnesium sulphate ($Mg\ SO_4$) into magnesium (Mg) and sulphuric acid (SO_4). This explanation is according to Arrhenius's Theory of Electrolytic Dissociation. Ions, like suspended colloids, are of two kinds, namely, positive and negative, and it is customary to speak of the "electric charge" of an ion—that is, a positive ion as carrying a positive charge and a negative ion a negative charge.

The important fact in this connection is that the positive ions are mainly responsible for the precipitation of negative colloids and *vice versa;* thus it is the chlorine ion in common salt which mainly effects the precipitation of colloidal ferric hydrate. The chlorine ion carries a negative, the colloidal ferric hydrate a positive charge.

An example of the precipitation of negative colloids by electrolytes is the formation of bars at the mouths of rivers carrying colloidal

clay in suspension. While the formation is due in a large degree to the checking of the velocity of the current, which causes the coarser particles to precipitate, it is the presence of electrolytes in the sea water which causes the precipitation of the extremely finely divided clay. These facts are well established; the mechanism of coagulation has not yet been completely elucidated, although chemists have advanced interesting hypotheses.

The importance of the law of electrolytic dissociation in this connection lies in the fact that, in a suspension of colloidal ferric hydrate, the speed of precipitation, other things being equal, is proportional to the concentration of the negative ions in the ground-water under consideration. The precipitation is little accelerated, if at all, by the presence of positive ions; for example, it is the amount of the carbonic acid ion in calcium carbonate, not the amount of the calcium ion—the number of the chlorine ions in magnesium chloride—not the amount of the magnesium ion—which does the work. In the latter case the magnesium ion is charged with positive electricity to double the extent that the chlorine ion is with negative electricity, because one magnesium ion is equivalent chemically to two chlorine ions.

Freundlich* has determined the relative amounts of various electrolytes required to coagulate colloidal ferric hydrate, as follows:

Salt.	Coagulation concentration, in milli-equivalents per liter.
Potassium sulphate	0.41
Magnesium "	0.93
Sodium chloride	9.3
Barium "	9.6
Potassium nitrate	11.9

This shows that, other things being equal, waters containing sulphates precipitate iron much more rapidly than those containing the same amounts of chlorides or nitrates.

Schmidt and Bunte† have made experiments to determine the effect of various salts on the speed of precipitation of iron from iron-containing waters, and have obtained results in accordance with Freundlich's observations. For example, they found that carbonates

Zeitschrift für Phys. Chem., 44, 135 and 151 (1903).
†*Loc. cit.*

aided precipitation more than sulphates, and sulphates more than chlorides. This explains why the waters of the North German plain, containing much carbonate and considerable sulphate, are more readily purified (leaving the effect of vegetable matter out of consideration) than the soft waters of the New England coast.

Presence of Alkalis.—After the iron is oxidized, alkalis assist the precipitation of iron as do other electrolytes, by increasing the concentration of negative ions.

Agitation and Friction.—Many observers have noticed that if one of two bottles of an aerated, iron-containing ground-water be shaken, it will deposit iron more quickly than the sample which is not shaken. Perhaps shaking serves to bring the particles together just as the churning of cream causes the globules of fat to separate from the milk. If inert suspended matter be present during the shaking, the precipitation is hastened. Likewise the passage, through a layer of clean, pure, quartz sand or glass beads will often cause the sudden precipitation of the iron.

Contact with Filter Material or Precipitated Iron Hydrate.—It has been the common belief of Piefke and many others that it is the contact with filtering material and not the size of the same which precipitates and retains the ferric hydrate; for example, many European deferrization plants have filtering layers composed of sand which is coarser than 1 mm.; a material seemingly coarse enough to pass quite coarse precipitates. This belief is tenable. The vital thing is multiplicity of surface contact. In this connection may be mentioned the well-known efficiency of gravel-roughing filters* for the removal of colloidal suspended clay from the waters of muddy rivers.

If a little precipitated ferric hydrate be added to a sample of water just about to precipitate its iron, the action will be accelerated to a marked degree. This is another reason why filters are more efficient after the filtering material itself has acquired a layer of iron hydrate than before. The action of the iron hydrate has been shown by Dunbar and others to be purely a physical one. It is really a porous, spongy mass which increases the effective contact area exposed to the water passing through the filter; or if added directly to a sample of water, it exposes a very large surface to the water and hastens the

*Reports of Water Supply Department of Philadelphia, Pa.; Ad. Kemna, *Engineering News*, 59, 328.

precipitation in this way. Again, one may conceive of it as a "starter," that is, multitudinous nuclei around which the fine particles of suspended colloidal ferric hydrate cluster, thus helping to neutralize the electric charges which keep the hydrate in suspension.

Temperature.—Graham and others have stated that temperature affects the coagulation of colloids in concentrated solutions only.

Schmidt and Bunte* have observed that while an increase in temperature accelerates the liberation of carbon dioxide, and therefore increases the speed of oxidation, the direct effect of temperature on the precipitation of iron from waters having the same amounts of carbon dioxide could not be determined. At Knoxville, Tenn., the writer has observed that the coagulation of the finely divided clay in the Tennessee River water by sulphate of alumina is much more easily accomplished in summer than in winter; still, other conditions, such as the character of the suspended matter and the concentration of salts (electrolytes) in the water, may not be constant throughout the year, and may offset the apparent effect of temperature.

Doubtless ferric hydrate will coagulate a little faster at higher than at lower temperatures, but this effect is too slight in the case of a dilute solution of fairly uniform temperature, like ground-water, to be of any considerable importance from an engineering standpoint.

Humus Compounds.—Humus is the active principle of leaf-mould, and the name has been given to a mixture of various substances of indefinite composition which have resulted from the decomposition of organic matter, in the partial or complete absence of oxygen. They may be considered to be the products of Nature's septic tank. It is necessary to distinguish the main classes of humus matter from one another; even in recent literature, statements have been made which show a lack of appreciation of any difference in humus compounds. Some of these bodies are albumens; some are gums; some are organic acids; some are acids in combination with calcium, magnesium, iron, etc.; and some of the bodies combine with iron and form a brown-colored compound. From a water-purification standpoint, however, there are three classes, namely, those in solution, those in colloidal solution, and those in colloidal suspension.

The first class probably affects the deferrization process but little. Not much is known of this class, and their existence is doubted by

Loc. cit.

many chemists. They may impart taste or odor to a water, which taste or odor can be removed by aeration, ozonization, or absorption.

The second class, humus matter in colloidal solution, is a great hindrance to the precipitation of iron. These gelatinous or gummy bodies are not precipitated by aeration. They remain in solution indefinitely until absorbed by suspended matter or by the action of filtering material, or until they are destroyed by bacterial action. What is worse, a very small amount of these gelatinous bodies will hinder the precipitation of a larger amount of iron. They are more apt to be present in seepage water from near the surface than in the dark brown water of deep wells, leading one to believe that the more recent the decomposition of vegetable matter, the larger will be the proportion of these humus bodies in colloidal solution. They yield readily to treatment with ozone.

The third class of humus matter is also important. It has been known for a long time that the coloring matter in the Dismal Swamp (Virginia) water when exposed to air would coagulate and precipitate spontaneously. Most of the surface water in the Bavarian forest has a brown color, but after a short run in the brooks and rivers it becomes perfectly clear.*

The writer has observed the same phenomenon in the Adirondacks. Such waters are apparently clear, but the coloring matter is really in colloidal suspension.

Recently, Wernike and Weldert† have shown that when the dark brown, Artesian well water from the vicinity of Posen (Table 11) is mixed with the iron-containing, shallow well-water supply of the city, the color and iron are precipitated immediately, enough oxygen being present to oxidize the iron. The water of some of the wells supplying Kiel contains both coloring matter and iron which when exposed to the air precipitate spontaneously.‡ The precipitate is in the form of dark brown, coarse flocs which contain humus matter and iron only. In this case, the different waters were originally in the ground in layers, and were mixed below the surface rather than above, as at Posen.

The Effect of the Relative Concentrations of Positive and Negative Colloids, Respectively.—The best explanation of the above-mentioned

*H. Blücher, "Das Wasser," p. 119.

†*Loc. cit.*

‡Professor Dr. B. Fischer, *Hyg. Rund.*, 17, 1089.

TABLE 11.

Proportions in Mixture.		Filtered Water One Hour After Mixing.					
Shallow well water.	Artesian water.	Clearness.	Appearance: Transparency, in centimeters.	Color.	Chlorine.	Iron. Fe_2O_3.	Oxygen consumed.
Ferruginous.	Colored.						
cu. cm.	cu. cm.						
900	100	turbid	8	yellow brown	34	16.	40
800	200	slt. turbid	12	" "	48	6.	28
700	300	v. slt. turbid	15	yellow	68	2.1	15
600	400	clear	over 18	very slt. yellow	82	0.4	13
500	500	"	" 18	almost none	99	0.1	12
400	600	"	" 18	" "	112	0.1	12
300	700	"	" 18	" "	127	0.0	14
200	800	"	" 18	" "	148	0.0	23
100	900	"	" 18	very slt. brown	161	0.0	48
75	925	"	" 18	slt. brown	165	0.0	96
50	950	"	" 18	" "	171	0.0	98
25	975	"	" 18	dark brown	173		169
10	990	"	" 18	" "	175		207
Shallow well water				slt. yellow	18	18.	24
Artesian water				dark brown	178	0.9	246

precipitation phenomena is that the brown coloring matter is a negative, suspended colloid and the ferric hydrate a positive, suspended colloid; that is, their electric charges have opposite signs, and, when mixed in proper proportions in the presence of oxygen, precipitate one another, probably because the electric charges balance and neutralize. Whatever may be the explanation, it is well known that either the coloring matter or the colloidal ferric hydrate will stay in suspension in pure water indefinitely when kept separate, but that when mixed in certain proportions, they will mutually precipitate each other. From a practical standpoint, the proportions of the mixture may vary considerably without affecting the general result; that is, the line separating mixtures which will precipitate from those which within a reasonable time will not, is a broad one, more of a zone than a line, as the experiments, shown in Table 11, by Wernike and Weldert with Posen water will illustrate.

The coloring matter at Posen is derived from brown coal deposits which underlie the city and its vicinity. Weldert prepared colloidal suspensions of brown coal and ferric hydrate, respectively, and found that they mutually precipitated one another, as did the natural waters containing them.

Professor Fischer, of Kiel, could not cause the precipitation of colloidal ferric hydrate by solutions of peat. Apparently he did not distinguish the younger and fresher coloring matter in peat from that in brown coal. The former is probably in colloidal solution, and, therefore, hinders the precipitation of the colloidal suspensions of either ferric hydrate or color, which may be present.

The reverse of the above, namely, the use of colloidal ferric or aluminum hydrate for the removal of coloring matter from water and other liquids, has been practiced for centuries. Alum, which forms aluminum hydrate when added to colored alkaline water, is one example, and Proskauer's proposal to treat the dark brown deep-well waters of Germany with colloidal ferric hydrate, prepared by passing an electric current through iron plates immersed in the water, is another.

During the writer's experience at Reading, Mass., colloidal iron was added to the water with the hope that it would cause the precipitation of the organic matter, and thus permit the precipitation of the iron contained therein. The colloidal iron hydrate thus prepared did

combine with the organic matter, but no advantage was gained; indeed it was necessary to add enough iron hydrate to remove the organic matter by the absorbent action of a large bulk of precipitated hydrate. When this was done, the iron present in the ground-water could be precipitated, but under these conditions the filter clogged too rapidly for economical operation. The added colloid had the wrong sign for the removal of the iron and had little effect upon the organic matter, because most of the latter was not in colloidal suspension. Since the preparation of this paper was begun there has appeared a serial article by M. Henri Schwers, of the University of Liége, in which it is stated that this compound of iron and organic matter is also in colloidal suspension, an interesting fact, if true.*

Use of Ozone as an Aid to Deferrization.

Ozone is an active oxidizer and also causes the precipitation of iron from ground-water in a marvelous manner, first, by rapid oxidation of the iron; and, second, by the destruction of the organic matter in colloidal solution which interferes with precipitation.†

Hitherto its cost has been prohibitive, but recent developments in France are so promising that it is probable that the cost will be reduced within practicable limits in the near future. Ozonized water, like aerated water, must be filtered before distribution.

Deferrization by Organisms.

Organisms grow in ferruginous ground-water the same as in surface water, but the usual absence of light favors the growth of a special group of fungi, the so-called iron bacteria. Among these are *Leptothrix ochracea, Crenothrix polyspora, Gallionella ferruginea, Clonothrix fusca, Anthophysa vegetans* (Müller), and *Actineomycetes.* Of these the more important are *Leptothrix, Crenothrix,* and *Gallionella.* These latter consist of a single row of cells surrounded by a gelatinous sheath. The iron deposits on the sheath, and, in the presence of oxygen, the organisms appear like irregular threads of ferric hydrate, at the ends of which are the colorless young cells partly surrounded with their nearly colorless sheath. Unlike algæ or diatoms they contain no chlorophyl. They grow in the absence of light; and apparently

*Révue d'Hygiène, 30, 108.

†G. A. Soper, M. Am. Soc. C. E., *Engineering News,* 42, 250.

they grow best in the presence of iron and small amounts of organic matter and oxygen. They are very hardy and can survive in the depths of the earth, from which they can get into good well-water.

This class of organisms undoubtedly does cause the precipitation of iron from ground-water, but it is natural to ask whether the iron is a food for the growth of the fungus and is subsequently excreted by it, or whether it acts like finely divided, gelatinous, suspended matter and causes the precipitation of the iron by contact and absorption.

M. Schwers has studied the formation of natural ochres in Belgium, and his results are most interesting and instructive.* He examined 208 deposits of different colors and structures, all mixed with more or less organic débris, bacteria, diatoms, filamentous bacteria, etc. He found ferruginous bacteria in 160 of the 208 deposits studied. Of these, 51 contained pure cultures of *Leptothrix* and 18, pure cultures of *Gallionella*. *Crenothrix* was not found. Besides the ferruginous bacteria, green algæ were found in 34 and diatoms in 41 deposits; while 21 deposits contained only green algæ and 22 only diatoms. It is quite evident from these observations that organisms other than filamentous bacteria and organisms whose growth is surely independent of the presence or iron, play parts in the deposition of iron from water containing it. M. Schwers believes, as the result of his observations, that the deposits of iron on the sheaths of these organisms is purely mechanical, the result of absorption and surface contact.

In May, 1904, the writer made observations at Reading, Mass., where the gallery water was treated by passing it through a box containing iron nails and thence directly to a sand filter with wooden sides. The filter had an area of 18 sq. ft., and the rate of filtration was 6 000 000 gal. per acre per diem. The period of contact between the water and the metallic iron was at least 1 min. The water was not aerated, although it contained about 1 part of oxygen per million. The filter removed the iron almost completely. Examination showed that the sides of the box containing the iron, the wooden walls of the filter, and the surface of the sand were covered with a growth of *Leptothrix ochracea*, in some places 2 in. thick. An aerator was interposed between the box containing the iron and the filter. On May 6th, with the aerator in service, the water in the filter contained 5.27 and the

*Révue d'Hygiène, loc. cit.

filtered water 1.18 parts of dissolved oxygen per million; soon, however, the growth of *Leptothrix* disappeared, probably because of the excess of oxygen present, and the efficiency of the filter declined rapidly. On May 28th, the well-water contained 1.5 parts and the filtered water 1.6 parts of iron per million. The amount of carbon dioxide during these experiments was about 20 parts per million, on the average. Microscopical examination of the *Leptothrix* and inspection of the surface of the sand showed that most of the iron had adhered to the *Leptothrix*.

Without going too deeply into the details, it may be stated that no one has yet succeeded in growing any of these organisms in the total absence of iron, and there are analogous organisms (*Beggiatoa*) which precipitate sulphur from ground-water containing it. On the other hand, there is an apparent precipitating effect independent of the biology of the organism and due to the physical action of the gelatinous sheaths of the organisms.

It is also possible that the same organic matter which holds the iron in suspension is a food for these bacteria, and that the removal of the food permits the precipitation of the iron. If this be true, precipitation would take place, very logically, near or on the organisms.

Historical.

The difficulties experienced with waters containing iron and manganese have been already outlined. The whole object of the deferrization processes is to cause the oxidation and precipitation of these troublesome elements before the water enters the distribution system, and not afterward in the service pipes and on the laundered fabrics belonging to the consumer.

The processes which have been evolved are based upon very simple phenomena, and most failures have been due to the neglect of simple, elemental principles. The iron which appears at the surface of the ground in a soluble form must be converted into the insoluble form and be precipitated. The reagent used for this purpose is the oxygen of the air. The presence of carbon dioxide and gelatinous organic matter interferes with the oxidation and precipitation of the iron, while the presence of suspended matter and certain salts furthers it. In certain cases, simple aeration, subsidence, and filtration fail to remove the iron within a reasonable period. In such cases the water may be treated with a coagulant to hasten the precipitation.

To the writer's knowledge, the history of the development of the deferrization processes has never been written in English. Professor Dunbar, of Hamburg,* Baurath A. Thiem, of Leipsic, with Professor Fischer, of Kiel,† Director L. Darapsky,‡ and M. Henri Schwers,§ have given descriptions of many of the devices which have been used in Europe, and which, with many other published articles and the writer's observations, form the basis for the following statements.

Salbach, of Dresden,‖ must be credited with the first successful process on a large scale, although it was known for centuries that certain ground-waters could be freed from iron by allowing them to stand for a time in contact with the air.

In 1868, Salbach built at Halle an aerator and filter which produced a "perfectly clear and brilliant" effluent. The aerator was filled with coarse gravel, the filter with sand. Salbach's process, however, was not widely known and was never put into practice, although it is claimed by some that he advised the Charlottenburg Water Company, when the Teufelsee works were built in 1873-74.

In 1880, Anklam¶ aerated the ground-water at Tegel, and was able to remove the iron therefrom by subsequent filtration, while, without aeration, the iron ran through the filter. Anklam's aerator consisted of a step-like cascade built of wood, over which the water fell. Its total height was 2 ft., and it discharged directly upon the filter. In 1882, Wingen, without knowledge of Anklam's work, accomplished a similar result at Golgau; while Oesten and Proskauer,** from 1886 to 1890, conducted successful experiments with the Berlin ground-water, spraying the water from a height of 2 m. upon the water surface of a coarse gravel filter which removed the iron satisfactorily. In 1891, Piefke,†† published results of experiments at the Straulau works, in Berlin, where he used a coke instead of a gravel aerator. (See Fig. 5.)

Afterward, Wellman‡‡ used brick filling for the aerators at Charlottenburg, and, more recently, Beer§§ and others have used super-

Zeitschrift für Architectur- und Ingenieurwesen, 1898.

†*Deutsche Vierteljahresschrift für öffentliche Gesundheitspflege*, 29, No. 1.

‡*Gesundheits-Ingenieur*, 1904.

§*Révue d'Hygiène*, 30, 11, *et seq.* (1908).

‖"Bericht über die Erfahrungen bei Wasserwerken und Grundwasserversorgungen," Dresden, 1893.

¶*Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 35, 517.

**Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 34, 35.

††*Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 34, 56.

‡‡*Gesundheits-Ingenieur*, Vol. 17.

§§*Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 46, 681.

imposed bundles of wooden slats with equally good results, notably at Berlin.

All these processes consist of aeration followed by filtration. Sometimes a short period of sedimentation precedes the filtration.

Meanwhile another class of purification plant was being introduced, namely, the so-called "closed installation," first introduced into Germany, in 1897, by H. von der Linde and Dr. C. Hess, at München-Gladbach.* This system did not use aeration. It consisted of closed tanks or pressure filters filled with red beechwood shavings, which latter were impregnated with tin oxide. It was alleged† that the tin oxide furnished the oxygen necessary for the precipitation of the iron. These plants were quite extensively introduced, notably at München-Gladbach and at Düsseldorf.

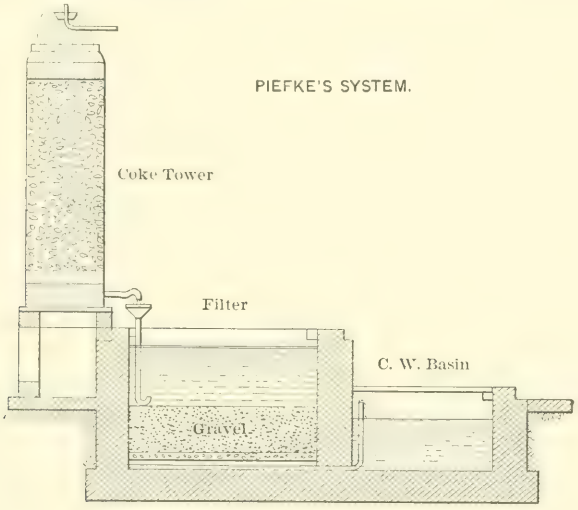


FIG. 5.

In 1902, Dr. Helm, of Danzig, replaced the tin-impregnated shavings in the von der Linde and Hess filter with magnetic iron ore. He obtained equally good results and without the danger of any increase in bacteria due to fermenting shavings. In 1903 Hertzberg, near Berlin, obtained corresponding results with a similar filter containing ordinary sand, and in the same year Director Bock, of Hanover,

Deutsche Bauzeitung, 32, 593.

†Schindowski in a dissertation (Leipsic, 1900, p. 24) proved that all closed filters fail to remove iron when oxygen is absent from the unfiltered water, regardless of the presence of tin or other oxides.

installed the new plant for that city in which untreated shavings were used, which, however, were prevented from becoming contaminated with bacterial growths by frequent sterilization with live steam.

In America, between 1890 and 1896, the Continental and other filter companies introduced several pressure filters, notably at Asbury Park, N. J. These filters consisted of pairs of closed tanks, the first filled with sand, the second with bone charcoal. At first the plants were installed without any provision for aeration other than that supplied by leaky pistons and suction mains and by the air which was used to agitate the filter material during washing. Another example is the plant at Atlantic Highlands, N. J. Experiments with the Asbury Park plant in May, 1895, by Dr. T. M. Drown showed that without aeration iron was incompletely removed after the capacity of the bone charcoal for absorbing iron had been exhausted, and aeration by a Pohle air-lift was accordingly introduced with good effect.

In 1896 Mr. H. W. Clark, Chemist of the Massachusetts State Board of Health,* made experiments with the water at Provincetown, Mass., a water which contained a great deal of organic matter and with which a sand filter operated fully as efficiently without as with thorough aeration. Mr. Clark found that, by passing the water through a filter filled with bits of iron, "the ferrous hydrate formed by the water and the iron filtering material would unite and effect a removal of the iron in the water to a remarkable degree, when after such treatment the water was passed through the sand filter." He also succeeded in effecting the same result by passing the water through fine coke which contained considerable iron. Mr. Clark believed that the almost complete coagulation of the iron during the latter experiment was due to the chemical action caused by the metallic iron contained in the coke, as well as to the mechanical action of the rough particles of the coke itself.

Mr. Clark also experimented with the use of potassium permanganate as an aid in the deferrization of the Provincetown water. The results were satisfactory; but the cost of the chemical prevented its adoption and practice. In addition to the experiments at Provincetown, Mr. Clark† made other experiments with the ground-water supplying Watertown, Marblehead, and Reading. The iron from the

Journal, New England Water-Works Assoc., 11, 277.

†Report, Mass. State Board of Health, 1899, pp. 537 to 553.

Marblehead water, as well as the color, was almost completely removed by simple aeration and filtration, even at rates of filtration as high as 15 000 000 gal. per acre per diem, while the effluent from the experimental filters at Watertown, even with long-continued aeration, always contained more than 0.2 part of iron per million. Mr. Clark attributed the difference to the presence of organic matter in the Watertown water, which held the iron in solution. It is an interesting fact that Mr. Clark notes that "better results were obtained in subsequent filtration by aerating in the manner described (simple spraying*)—which required but a few seconds—than by drawing air through the water for a much longer period." Experiments at Reading showed that lime would assist the removal of iron.

During 1905-1906, the writer made experiments with the Reading, Mass., water, treating it with metallic iron, first by a revolving drum filled with iron (Anderson process), and afterward, in the colloidal form (hydrate) by iron-plate electrodes immersed in the water and through which an electric current was passed. Although this process removed the iron in a fairly satisfactory manner, it was not recommended, both on account of its cost and because the treatment with clay and sulphate of alumina was more practicable under local conditions. Later experience leads the writer to believe that at certain times it would be impracticable to coagulate the iron by such treatment.

In 1898 Dunbar and Kryck† suggested intermittent filtration through coarse sand, especially where the available head is low, thereby avoiding double pumping, or where (as in small plants) the filter can be filled only occasionally. They believed that this type of filter could be used much more frequently than had been the case hitherto. In operation it is not filled above the surface of the sand, and special provision for aeration is unnecessary. It is especially adapted for use on small estates, in schoolhouses, etc.

In addition to the oxidation processes, there have been some plants built where chemicals are used to precipitate the iron. In 1891 Herr O. Kröhnke‡ added lime or sometimes lime and an iron salt to water and afterward removed the precipitated iron by a pressure filter.

In 1895, The Cumberland Manufacturing Company, of Boston, advised the Town of Reading§ to aerate the water from the filter gal-

*Report, Mass. State Board of Health, 1899, pp. 537 to 553.
†*Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 40, 548.
‡Dunbar, *Zeitschrift für Hygiene und Infektionskrankheiten*, 22, 118.
§Bancroft, *Journal*, New England Water-Works Assoc., 11, 294.

leries and, after the addition of lime and sulphate of alumina, to filter the same. The advice was adopted, and, in 1894, a "Warren" filter plant was installed. This was operated as designed until April 19th, 1906, when clay suspension was substituted for lime. A smaller plant using lime and sulphate of alumina was built at Keyport, N. J., at about the same time.

Recently, R. Gans* has suggested calcium aluminum silicate or artificial zeolite as a precipitant for iron-containing water. This is marketed under the trade name of "Permutit." The cost of the chemical, however, has been too high to justify its use in practice.

The most interesting recent development has been the discovery by Professor Wernike and Director Mertens, at Posen (Prussian Poland,† that a mixture of the iron-containing, shallow-well water (purified only with great difficulty by aeration and filtration) with a dark brown, deep-well water, could be filtered rapidly through sand, and that the effluent would be free from both iron and coloring matter which precipitated one another when the waters were mixed. This process has been adopted by the City of Posen, and the plant is to be installed immediately, with Jewell mechanical filters to remove the precipitate.

Since the preparation of this paper was begun, M. Henri Schwers‡ has suggested the use of ferric chloride as a coagulant. Kröhnke had previously used ferric chloride, but in conjunction with lime.

Table 12 summarizes the history of the various processes.

European Practice.

The writer has made frequent mention of European practice, especially of that which relates to the arrangement and type of works. It now remains to discuss some special features connected with their efficiency.

In North Germany and the Low Countries, generally speaking, the cities with ground-water supplies, built upon the alluviums of the various rivers such as the Oder, the Vistula, the Elbe, the Weser, and the Rhine, are provided with deferrization plants. Many of these cities, especially those in Germany, have been mentioned. The

Chemiker Zeitung, 1907, p. 355.

†*Mitteilungen a. d. Königl. Material Prüfungsanstalt*, 1907, No. 8.

‡*Révue d'Hygiène*, 30, 1908.

ground-water supplies of Berlin and suburbs are good examples of German practice, as illustrated by Table 13.

TABLE 12.—DEVELOPMENT OF THE DEFERRIZATION PROCESSES.

Process.	Inventor and system.	Date of Introduction.
Aeration followed by filtration through fine gravel or coarse sand..	Salbach (Gravel aerator)	1868
	Anklam (Cascade aerator)	1880
	Oesten (Spray aerator)	1890
	Piefke and Proskauer (Coke aerator)	1891
	Wellman (Brick-filled aerator)	1894
	Beer and Thiem (Wooden-slat aerator)	1902
Filtration without aeration by pressure filters.	American pressure filters (Sand and bone charcoal)	1894
	von der Linde and Hess (Tin oxide-impregnated shavings)	1897
	Helm (Danzig) (Magnetic iron ore)	1902
	Bock (Hanover) (Plain shavings)	1903
Treatment with metallic iron and filtration.....	Clark (Filtration through fine coke or iron chips)	1895
	Town of Reading (Electrolytic iron)	1904
Intermittent filtration through coarse sand...	Dunbar and Kryck	1898
Chemical treatment and rapid filtration.....	Kröhnke (Lime without aeration)	1891
	Cumberland Mfg. Co. (Reading) (Lime and sulphate of alumina with aeration)	1894
	Weston (Clay and sulphate of alumina)	1906
	Gans (Filtration through calcium aluminum silicate)	1907
	Schwers (Ferric chloride, slow filters)	1907
Precipitation by admixture of iron-containing water with colored deep well water, followed by rapid filtration.....	Wernike and Mertens (Posen)	1906

TABLE 13.—DEFERRIZATION PLANTS IN BERLIN AND SUBURBS.*

Municipality.	Consumption, in U. S. gallons per diem.	System.
Adlersdorf	715 000	Brick-filled aerator and sand filter.
Berlin (five works)	over 5 300 000	Wooden-slat aerator and sand filter.
Boxhagen-Rummelsbrug	1 375 000	Brick-filled aerator and sand filter.
Charlottenburg (four works)	6 000 000	" " " " "
Fürstenwalde-a-Spree	192 000	von der Linde and Hess.
Hohen-Schönhausen	31 600	Oesten.
Lichtenburg	2 200 000	Piefke.
Nauen	130 000	Oesten-Piefke.
Nieder-Schönhausen	658 000	Coke aerator and sand filter.
Pankow	823 000	O. Smreker.†
Potsdam (part)	.....	Thiem.
Remickendorf	1 238 000	O. Smreker.†
Spandau	3 020 000	Aerator and sand filter.
Tegel	3 565 000	" " " "

*Data furnished by Prof. Thum.

†Pressure filters.

Most of the other suburbs obtain water from one of the Berlin or Charlottenburg works. During 1905, the former city supplied 735 000 U. S. gal. daily, on an average, to these suburbs.

In Holland the water from the sand dunes is often delivered to fine sand filters by gravity. This water is apt to contain iron, but in many cases aerators are not necessary; the filtered water is excellent. Humus matter is the cause of most of the trouble with the Dutch dune-water supplies.

The plant at Hanover operates with a water containing an average of only 0.5 part of iron per million, also traces of manganese. The maximum amount of iron and manganese is about 0.7 part per million. Formerly, the works were operated without filters; but since 1904 the present installation has been in service. The filter plant, Plate CXLII, consists of 30 cylindrical units each 2 m. (6.6 ft.) deep, with diameters of 1.5 m. (5.0 ft.) at the top, and 1.35 m. (4.5 ft.) at the bottom. They are filled with red-beechwood shavings. These shavings are about 0.04 in. thick, 0.2 in. wide, and average about 1 ft. in length. The rate of filtration is 4.8 vertical in. per minute, or about the same as that used with American rapid filters. No aeration is provided except that which unavoidably takes place through joints in the suction pipe, in the collecting well, and through the piston rod glands. The filters were provided with apparatus for washing under pressure; this was found to be too expensive, however, and, at present, the shavings are removed from the filters, washed in masonry tanks, and then replaced in the filters. Three of the thirty tanks are cleaned daily; that is, each unit operates for ten days between cleanings. The writer saw one tank opened, and observed that the iron hydrate was quite well distributed through the mass of shavings, although it had collected most abundantly at the top. The freshly opened filter emitted an odor characteristic of decaying wood. This odor must be more marked in summer. After cleaning, the shavings are sterilized by steam; otherwise, the disagreeable odor would develop much sooner and would make it necessary to clean the shavings more frequently. The filters do not remove all the iron, and the deposit in the water-tower contains manganese.* The writer's observation leads him to believe that it is the light task rather than the suitability of the design, which makes it possible to furnish a reasonably satisfactory

*See p. 1385.

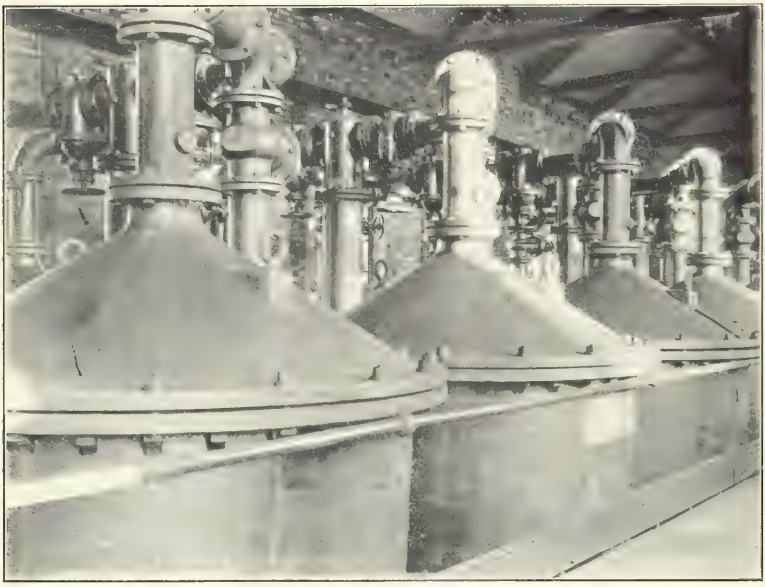


FIG. 1.—PRESSURE FILTERS, HANOVER, GERMANY.

FIG. 2.—APPARATUS FOR WASHING AND STERILIZING SHAVINGS, HANOVER, GERMANY.

water to the city. The aeration is accidental only, although oxygen is absorbed by the shavings during washing, and perhaps at other times.

The pressure-filter plant at München-Gladbach is similar to that at Hanover. Fig. 6 may help to explain this type of filter more clearly.

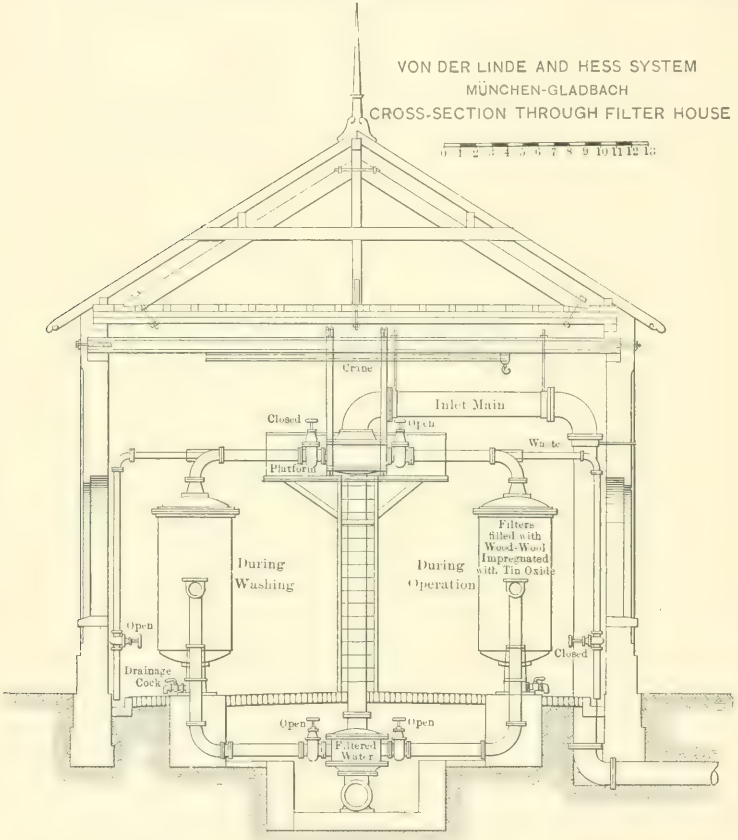


FIG. 6.

This plant has a nominal capacity of 76 500 gal. per hour. It consists of 11 filter tanks arranged in a circle; each tank is 4.8 ft. in diameter and 8.3 ft. high. The ground-water contains 2.3 parts of iron per million, and it is necessary to wash each filter thrice daily. Every two months the filter material is removed and cleaned in a washing-machine. Each filter holds 3 300 lb. of shavings impregnated

with tin oxide, and 550 lb. must be added after two months' use to make up for the shrinkage. The amount of wash-water is about 15% of that filtered, and the loss of head is about 10 lb.

The experience at Fürstenwalde-a-Spree is interesting. This plant also consists of pressure filters, after the von der Linde and Hess system, taking water directly from wells. The four filter tanks are filled with noodle-like shavings, and aeration is effected by admitting air into the suction pipe. After aeration, the water goes directly to the filters, and consequently the period of coagulation is very short. The works were put in service in 1905. At the beginning, the ground-water contained 2.0 parts of iron, but, in 1908, it contains only 1.4 parts of iron per million. Manganese is not present. In 1905 the amount of iron in the filtered water was 0.2 part per million; since then, the consumption has doubled and has reached about 317 000 gal. per diem. The iron in the effluent, however, has arisen to 0.3 part per million, and some of it precipitates in the water-tower and also in the distribution system. The rate of filtration is twice as high as that used for rapid filters in America, and when more than 265 000 gal. per diem are filtered, it is necessary to wash the filters twice a day. Improvements are suggested; one proposition is to bore the wells deeper; another is to install preliminary aerators or filters.

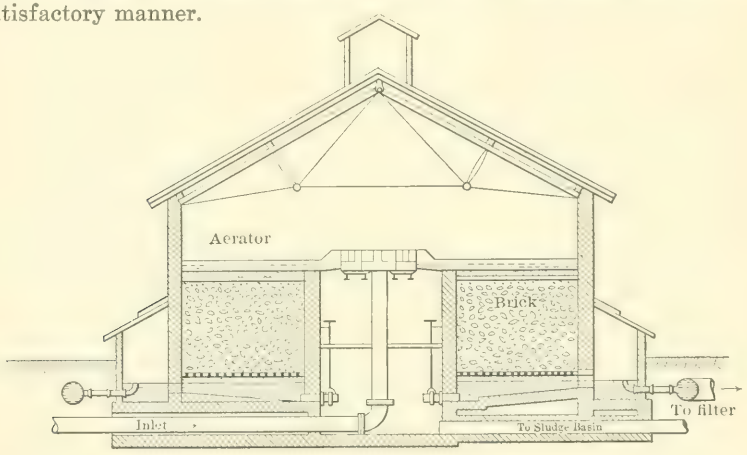
The experience at this plant illustrates very clearly the importance of the time element in the process. When more time elapsed between the wells and the filters, the iron was removed in a satisfactory manner. Recently, however, there has been a reduced efficiency.

The deferrization plant at Kiel* consists of aerator and filter, of which the aerator alone will be described. The water, 3 750 000 gal. per diem, is distributed uniformly over the surface of coke aerators which are 4.6 ft. square on the surface and 10 ft. thick. Eight aerator units are placed in a building immediately over the sedimentation basin. The coke rests upon an iron grating which admits air beneath the coke; numerous openings in the wall of the aerator also admit air. A channel, 2 ft. wide, delivers the water into an octagonal, cast-iron distributor from which it flows over eight weirs into the several compartments of the aerator. Each compartment has a main distribution channel from which leads a transverse gutter, and from this the water falls upon a sheet of corrugated metal, the corrugations of which are at

*Schillings Journal für Gasbeleuchtung und Wasserversorgung, 40 (1896).

right angles to the transverse gutter. Both the gutters and the furrows in the corrugated metal are pierced with holes. This arrangement effects a very uniform distribution of the water over the surface of the coke. A similar arrangement is used in connection with most of the aerators of the tower type in Germany.

The sedimentation basin beneath the aerator is provided with baffles, and the water delivered to the filters contains only from 15 to 25% of the initial amounts of iron. When the well-water contains 4 parts of iron per million, it is necessary to clean the aerator after 26 500 000 gal. have been passed through it. This plant works in a satisfactory manner.



CHARLOTTENBURG

FIG. 7.

The Charlottenburg aerators are filled with brick instead of coke. Fig. 7 shows the general arrangement of this plant.

In Germany the usual rate of application to the aerator is 16.6 cu. ft. per square foot per hour, although at Stargard, where the final filtration is quite rapid, this rate was found to be about 20% too high, and at Worms much higher rates are used.

The rate of filtration through final filters varies from 3 000 000 to 20 000 000 gal. per acre per diem. At Stargard it is 15 200 000 gal. per acre per diem, while, at Berlin, rates from 3 000 000 to 5 200 000 gal. per acre per diem are used. At Müggelsee a filter operated 417 days between scrapings, about twice the maximum period ever obtained when filtering surface water at half the rate.

The new filters at Hamburg, Fig. 8, are interesting in that they are exposed to the air as much as possible, especially during the idle period.* The water is distributed upon the surface of a coke aerator about 9 ft. thick, whence it passes through the coke into a small basin and thence through spray nozzles to the filters. The sand layer is only about 8 in. thick, and is composed of sand with grains 1 mm. in diameter, much finer material than is generally used in Germany. The surface of the sand is hardly covered with water, and during the idle periods the filters are drained. Ample aeration is provided, and the results have been excellent. The loss of head in the whole plant is about 22 ft.

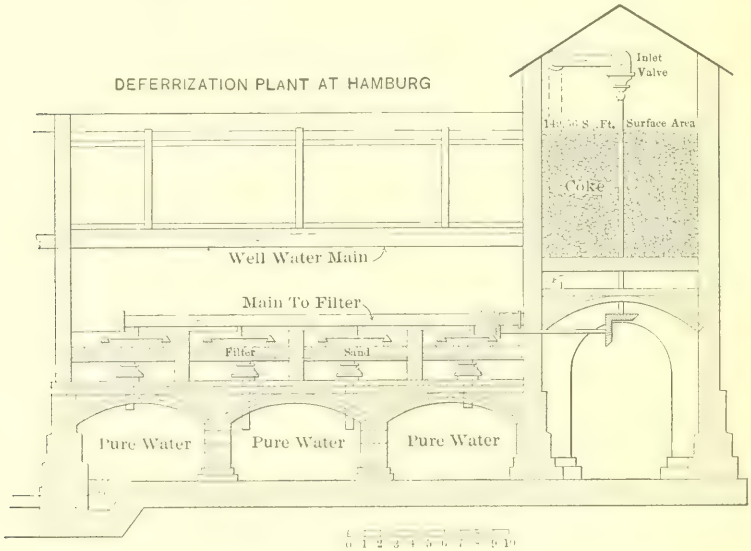


FIG. 8.

It has been customary to use quite coarse material for final filters. This has the advantage of causing the precipitation of the iron throughout the whole mass of filter material and allows a far more economical operation of the plant. On the other hand, these filters "break" quite frequently, and, where the filtered-water basin is small, may cause turbid water to be distributed; this is especially noticeable just before cleaning. This difficulty does not occur where finer sand is used, as at Berlin.

Schillings Journal für Gasbeleuchtung und Wasserversorgung.

New filters are being built at Harburg by Dr. G. Thiem, which are quite different in design from those at Hamburg. In January, 1908, these filters were not yet filled, and could be visited to great advantage. This plant has a capacity of 2 640 000 gal. daily, and occupies an area of about 432 by 100 ft., exclusive of the sludge basin, which is about 83 ft. long by 13 ft. wide. The plant consists of a wooden-slat aerator, graded gravel filter, and filtered-water basin. The main distribution channel of the aerator is of iron, but all other parts are wood. The distributing channel discharges into simple wooden gutters with perforated bottoms and notched sides, placed at right angles to the main channel. Beneath the gutters are four superimposed groups of wooden slats. Each slat is 3.2 in. wide and 0.32 in. thick, and is made of fir-wood. The slats, set on edge, are fastened together in grate-like bundles with distance pieces between. Each bundle is 3.2 in. thick, and the spaces between the slats are about 0.32 in. wide. The bundles are in four groups of four bundles each, and there is 8 in. between each group of bundles. The slats in any bundle are at right angles to those in the bundles immediately above and below. The aerator is in four units, and the rate of application is about 16.7 cu. ft. per square foot per hour, or 325 000 000 gal. per acre per diem. Each aerator is surrounded with louvers inclining inward, which admit air to the aerator and prevent the water from splashing outside. The aerators are covered with a brick building, as is the common practice in Germany, contamination of the water by dust being thus avoided. (Plate CXLIII.)

The water drops from one bundle of slats to another and finally into a small concrete basin from which it flows to the filter. There is a maximum of 6 ft. of water above the gravel in the filter. The two rectangular filters are built of concrete. By one long side of each filter runs a main collecting drain, which has a semicircular invert of about 14 in. radius, vertical sides, and is nearly 3 ft. deep. The whole bottom of the filter is covered with special hollow tiles cemented in place, giving it a corrugated appearance. Along each side of the main drain runs a wooden strip, about 3 in. wide and 0.5 in. thick, on which rests the oak blocks which form the cover of the drain. These blocks are 4 in. square on end and 4.3 ft. long. Resting on the bottom is a graded layer of coarse gravel, 20 in. thick including the tile, and above this is a layer of fine gravel 45 in. thick. The gravel is of six sizes as given in Table 14.

TABLE 14.

Layer.	Diameter of Particles, in Millimeters.	Thickness, in inches.
1	50	
2	25	
3	12–14	20
4	10–12	
5	8–10	
6	8–14	45

The difference in level between the water in the distributing channel and that in the filtered-water basin is about 17 ft. When the filter is to be cleaned, the water above the gravel is made to rise about 2 ft. higher than when filtering; a large flap-gate connected with the underdrains is opened suddenly, and the rush of water through the filter flushes the iron hydrate into the sludge basin. The sludge soon settles and is shoveled out.

Director von Feilitzsch, of Brunswick, has used this sludge to purify illuminating gas in place of the ferric oxide produced by rusting metallic iron.* The results have been satisfactory, and, furthermore, a waste product has been utilized.

The filter at Leipsic was built of layers of graded gravel of the thicknesses and sizes given in Table 15:

TABLE 15.

Thickness.		Diameter of Particles, in Millimeters.
Millimeters.	Inches.	
50	2.	50
325	13.	12 to 14
325	13.	10 " 12
325	13.	8 " 10
325	13.	6 " 8
60	2.4	8 " 10
60	2.4	10 " 12
60	2.4	12 " 14
60	2.4	25
60	2.4	50

The plant at Worms is similar in arrangement to that at Kiel.† It consists of three aerators, two filled with coke and one filled with stone, three filters, and a clear-water basin. There are also small subsiding

*Schillings Journal für Gasbeleuchtung und Wasserversorgung, 49, 714 (1906).
†Gesundheits-Ingenieur, 31, 364 (1906).

FIG. 1.—COVERED FILTERS AND PUMP-HOUSE, BERLIN, GERMANY.

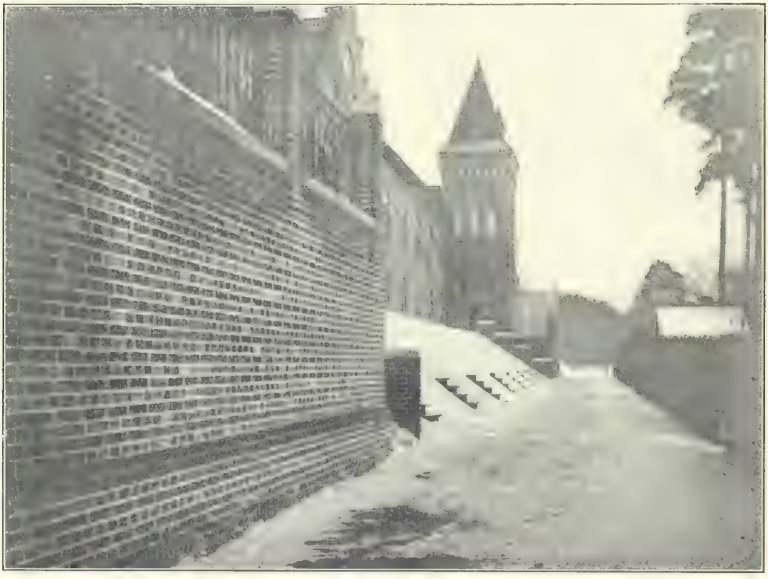


FIG. 2.—AERATOR HOUSES, BERLIN, GERMANY.

basins beneath the aerators. The aerators have a total surface area of 591 sq. ft. It has been proved by experiment that 1 sq. ft. of aerator surface can treat 4.56 gal. of water per minute in a satisfactory manner. This means that 348 sq. ft. of aerator surface would treat the 1 628 gal. of water per minute (2 342 000 gal. per diem) supplied to the aerator. Two aerator compartments are sufficient for this quantity of water, one being held in reserve. This rate is equivalent to 37.4 cu. ft. per square foot per hour, more than twice as great as the customary rate in Germany. The three filters have a total area of 0.118 acre. A rate of filtration of nearly 10 000 000 gal. per acre per diem can be used. One of the three filters is held in reserve.

The interesting plant which is to be built at Posen has already been mentioned, and it is believed that what has been written describes the main types of European deferrization plants.

It is, of course, difficult to compare the efficiency of these types on account of the varying conditions under which they operate. For example, almost any filter would operate successfully with water like that at Hanover, while there are waters which no existing German plant would adequately purify. The system of Oesten usually fails when the amount of iron is large. It is generally admitted that oxygen is a necessity for the treatment of water, whether it be present by accident or design. Other things being equal, the devices which afford the longest period of contact of the water with the filtering material are the most successful.

Quite recently, Schlegel and Merkel have experimented with the various processes and their feasibility for use with the ground-water in the vicinity of Nüremburg, and have published the following results.*

"1. The process of A. Bock (Hanover), which is patterned after the von der Linde and Hess system, maintained its efficiency without change, but removed only from 43 to 61 per cent. of the iron; the amounts of iron in the filtrate range from 0.16 to 0.62 part of iron per million, and are not parallel with the amounts present in the unfiltered water. The bacteria are markedly increased and the filter material has to be sterilized frequently.

"2. The 'Agga-Verbund' filter (artificial sand-stone filter) removed the iron in a satisfactory manner and without any marked increase in the number of bacteria, but its capacity was diminished to half the initial capacity after three days of use, so that it had to be cleaned frequently, thereby making the cost of operation prohibitive.

Chemiker-Zeitung, 31, 396.

"3. The deferrization plant at the Truant School in Brunnesburg where the water falls upon a cascade-like aerator from which it flows upon a simple sand filter with 1-mm. grains, effects a most satisfactory removal of the iron without any bacteriological complications."

Schlegel and Merkel also showed that aeration was not necessary, because the water contained 0.39 part of oxygen per million, and the iron could be removed by simple filtration through sand. In some localities they found that the iron was dissolved in the water in the colloidal form, being kept in solution by organic matter.

"In this case the ferrous hydrate is retained in the sand filter by adhesion or absorption, and afterwards becomes oxidized by the oxygen dissolved in the water. The precipitated ferric hydrate helps the precipitation of the colloidal iron by its coagulating action on the one hand, and, on the other hand, by diminishing the sizes of the pores of the filter, thereby preventing the passage of the fine precipitate."

M. Henri Schwers has made extensive experiments with the water at Turnhout, Belgium.* This water contains organic matter in combination with iron, as well as iron in the form of hydrate. The iron cannot be completely removed by aeration and filtration. He used many different coagulants to assist the precipitation of the iron. Among these were chlorinated lime, sulphate of alumina, ferric chloride, and a mixture of chlorinated lime and ferric chloride known as "ferro-chlor" (Duyk). He used these chemicals singly and in combination, adding them to the aerated water in all cases, and came to the following conclusions:

1.—Chlorinated lime alone is of no especial value as a coagulant in practice.

2.—Sulphate of alumina alone is only of small value as a coagulant in this case.

3.—Ferric chloride is a very efficient coagulant.

4.—The use of chlorinated lime with sulphate of alumina does not increase the efficiency of the latter. The mixture known as "ferro-chlor," has shown no superiority over sulphate of alumina or the ferric chloride used during the experiments.

5.—None of these agents is capable of producing immediate precipitation. Even the ferric chloride with chlorinated lime does not produce precipitation until after twenty-four hours, and this only when conditions are most favorable.

*Révue d'Hygiène, 30, 297 (1908).

6.—The most effective agent for the removal of iron from this kind of water is the deposit of ferric hydrate which accumulates in the filter. The function of organisms in the removal of iron is only a secondary one.

7.—The plan to follow at Turnhout is to depend upon the coating of ferric hydrate on the filter aided by coagulants, especially ferric chloride.

In this connection it is interesting to note that about one-tenth as much coagulant was required with well aged filters as with those freshly filled with clean sand.

The pressure filters of Germany are not to be recommended, either for efficiency or design; in these respects they are far inferior to the rapid filters of American design; but the coarse filter used in so many works is a most valuable device. In it the iron is retained throughout the whole depth of the bed until flushed out. This flushing is accomplished at small expense whenever necessary, and, although the construction cost of these filters is rather high, the cost of filtering the water is very low.

The writer cannot see the necessity for the great losses of head—more than 20 ft. in many cases—which occur in most German works. It would seem possible to get the same amount of aeration and surface contact without necessitating so much extra pumping.

Generally the European filters operate in a very efficient manner; when they fail, it is usually because of lack of oxygen, time for coagulation, or the presence of humus matter. German engineers have attacked the problems presented in a thorough manner, and have worked out practical solutions for all ferruginous waters with the exception of those containing interfering amounts of humus matter.

Experience at Reading, Massachusetts, and Other American Water-Works.

Experience at Reading, Mass.—The quality of the Reading water, the nature of its source, and the different methods of treatment which have been proposed for it, have already been mentioned. As has been stated, the chief obstacle was the presence of organic matter. Evidently interfering amounts of organic matter were not present in 1893, when Mr. Desmond FitzGerald studied the problem on behalf of the town, although he remarks that "the Reading water is no ordinary

fluid." In this report Mr. FitzGerald makes no mention of organic matter, and recommends aeration and subsidence in a reservoir holding two or three days' supply followed by filtration as a method of treatment which will make the water perfectly satisfactory. Two years later, however, the Massachusetts State Board of Health published results of experiments and also its advice to the Town of Reading.* It stated that:

"In one case at least, when the iron was only 0.14 parts per 100 000, aeration in this way (that recommended by Mr. FitzGerald) had but little effect on the iron. The failure to remove the iron in this case seemed to be connected with a considerable amount of organic matter in the water, which prevented the precipitation of the iron after oxidation."

At this time the State Board of Health was not in a position to recommend any method of deferrization; it advised rapid filtration as a temporary measure, and connection with the Metropolitan Water Supply in the future.

It should be noted in this connection that Mr. FitzGerald's belief in the feasibility of purifying the water was established while the meadows were artificially flooded by a dam. Subsequently the dam was removed, and it may be true that the alternate draining and flooding of the river-bottom soil promoted the solution of organic matter and iron from it. It is certainly true that the worst water in the memory of Mr. F. F. Strout, who has had charge of the pumping-station since the works were put in operation, occurred during the winter of 1907-08. This water contained a large amount of iron.

The Massachusetts State Board of Health† made extensive experiments at Reading, and advised the addition of metallic iron by contact with the water, followed by filtration through sand at a rate of 6 000 000 gal. per acre per diem. This method was carefully tested by the Reading Water Commission, but conditions were found when even this method proved inadequate. Iron was then added electrolytically, and while this process greatly increased the mass of precipitate and helped coagulation in this way, there were short periods when it failed, as has been already mentioned.

It was observed sometimes that long periods of subsidence effected little or no change in the character of the water, and, at times, the

*Report, Mass. State Board of Health, 1895, pp. 41 to 43.
†Report, Mass. State Board of Health, 1899.

iron could not be retained even by fine sand filters operating at rates of less than 6 000 000 gal. per acre per diem, especially if the water was aerated.

It would seem as if all the different investigations of the problem had been carried to logical conclusions, but that changes in the character of the water had revealed the growing importance of the organic matter in each case. Therefore, methods which proved adequate during one period of investigation were found to be impracticable on subsequent trial. Lime precipitates the objectionable organic compound* and helps to precipitate the iron. It was used for ten years to treat the water, sulphate of alumina being used with it. However, lime increased the hardness, and it was desirable to devise a method of purification which would obviate its use. It occurred to the writer that if colloidal suspended clay having a negative electric charge could be precipitated by colloidal hydrate of alumina having a positive charge, as is the case in all water-purification plants for the removal of suspended matter from muddy river waters, then colloidal ferric hydrate, which has the same charge as the alumina, would be precipitated by clay. It also seemed reasonable to ask that if the water of muddy rivers which often suffer large additions of highly colored swamp water is almost invariably low in dissolved color, why would not suspended clay absorb the color, that is, the organic matter in the Reading water interfering with the removal of iron? The writer remembered in this connection that the Mississippi River water below its junction with the highly colored Yazoo River was practically free from color when its suspended matter was removed by passage through a Pasteur filter, that is, it contained little or no dissolved color. Mr. H. W. Clark notes that at the Hanley, England, sewage-disposal plant, the mine wastes containing iron and the pottery wastes containing clay, by mutual coagulation, assisted materially in the precipitation of the sewage.† Accordingly, on April 19th, 1906, clay was substituted for lime at Reading, and ever since (with the exception of a short period during 1908) the plant has operated without lime.

Lime increased the hardness of the water about 200%, that is, from about 27 to about 81 parts per million, while no increase was noticed

*Report, Mass. State Board of Health, 1899.

†*Journal*, Assoc. Eng. Soc., 1908.

in the case of the water treated with clay and sulphate of alumina. It required about 150 lb. of lime daily to treat about 200 000 gal. of water. At the present writing, less than 15 lb. of clay are being used for the same amount of water; furthermore, the amount of sulphate of alumina has been reduced from more than 3 gr. to about 1 gr. per gallon. The filters in use with the lime and sulphate of alumina were "Warren" filters filled with sand having an effective size of 0.90 mm. The floc formed with the clay is finer-grained than that formed with the lime, and is not so easily retained by this coarse sand. The speed of coagulation is slower. However, the treated water passing through a fine sand filter was always satisfactory in every respect. Money has been appropriated for the remodeling of the plant, and modern mechanical filters with finer sand, preceded by either a roughing filter or a subsiding basin of increased size, will probably be built shortly.

Roughing Filters vs. Subsiding Basins.—Experiments were conducted at Reading during 1908 to determine the relative efficiencies of subsiding basins and roughing filters of the same construction cost. After being treated with clay and sulphate of alumina the water was passed through a subsiding basin having a capacity equivalent to about 1 hour's flow, and then into a barrel filled with No. 2 broken stone. Experiment No. 1 extended from March 16th to April 22d. The rate of filtration was 50 000 000 gal. per acre per day, and daily analyses showed the amounts of iron in the raw water, the effluent of the coagulating basin, and the roughing filter, respectively, given in Table 16.

TABLE 16.—Amounts of Iron, in Parts per Million.

Sample.	Experiment No. 1.	Experiment No. 2.
Raw water	2.40	2.28
Coagulating basin effluent	3.24	5.00
Roughing filter effluent	1.70	2.08

The filter was then shut down until May 25th, on account of lack of time to supervise it; after which date it was operated until June 8th, when it was washed. The results are given in the final column of Table 16.

During the last three days of this experiment the amount of iron

in the effluent of the roughing filter was greater than in the influent, showing that the suspended matter was being dislodged from the stones and carried away with the effluent. After washing, the efficiency of the device was restored. The results are a little misleading because the engineer of the pumping-station, Mr. F. F. Strout, who made the determinations, could make but one or two determinations of samples from each source daily, and apparently samples of the coagulating-basin effluent were taken at a time when they contained an abnormal amount of suspended iron. It is apparent from observations made at the plant, by the writer, that little sedimentation of iron takes place in the coagulating basin, although probably it is not true, notwithstanding the results, that there is no increase in the amount of iron after the passage of the water through the basin. The experiments are still going on, but the results cannot be stated definitely at this time. So far they show that more than 29% of the iron present in the raw water and more than 47% of that present in the coagulating-basin effluent is removed by the roughing filter, while its removal by the coagulating basin is a negligible quantity. What is fully as important as the actual amount of iron removed, is the condition of that remaining in suspension after it passes through the roughing filter. This is so well coagulated that passage through ordinary filter paper reduces the amount of iron to less than 0.1 part per million. The period of contact between the water and the filtered material of the roughing filter is about 15 min. A coagulating basin of the same construction cost would provide about 1 hour's storage, which the evidence at hand indicates would prove to be less efficient than the 15-min. storage in a roughing filter of the same construction cost in contact with its filtering material.

It should be noted that this roughing filter was filled with uniform broken stone and would not be as efficient as one of graded gravel, like those described in another part of this paper.

One advantage of a roughing filter at Reading would be the reduction in the amount of chemical treatment necessary, with clay and sulphate of alumina. It is believed that for a greater part of the year roughing filters followed by rapid filters would remove the iron in a satisfactory manner without chemical treatment.

Deferrization Plants in America.—Few deferrization plants have been built on this continent. Waters containing iron are too often

avoided. The plant at Asbury Park was the first one of importance; this was in use in 1894. Since then plants have been built at Far Rockaway, N. Y., Red Bank, N. J., West Superior, Wis., Richmond, Mo., Reading, Mass., Keyport, N. J., Garrettsville, Ohio, and a few other places. These plants are of three general types, namely:

1.—Aerators and pressure filters.
2.—Aerators and gravity filters.
3.—Gravity mechanical filters using coagulants.

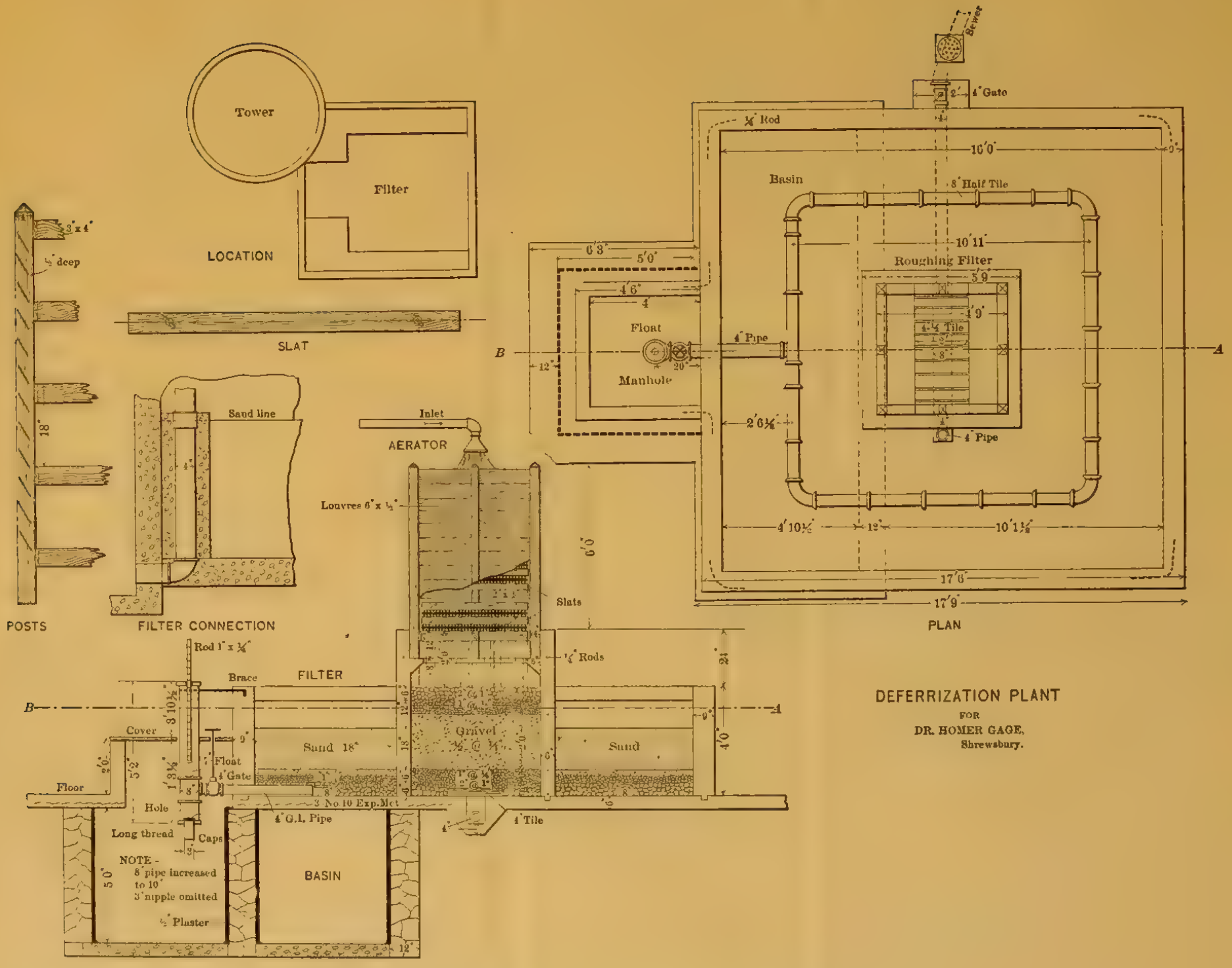
The plants at Far Rockaway and West Superior, designed by Mr. Allen Hazen, resemble the Oesten system more than any other.* The water is aerated by spraying and is then filtered through rather fine sand. The filters designed by Oesten contained coarser filtering material. The plant at Richmond, Mo., designed by Wynkoop Kiersted, M. Am. Soc. C. E., resembles the West Superior plant in principle.†

William S. Johnson, M. Am. Soc. C. E., has recently designed a temporary arrangement for Dover, N. H.‡ Ferruginous spring water is aerated and purified by spraying intermittently upon the sandy soil in the vicinity of wells, from which latter the supply for the city is pumped. While a portion of the water is lost, 95% of the iron is removed, and without great construction cost. This temporary plant will be replaced shortly by a permanent filter.

Recently the writer designed a small deferrization plant for an estate in Shrewsbury, Mass. The water is from an artesian well in shale. It is very soft, is quite free from organic matter, and contains 18 parts of carbon dioxide, 3.2 parts of iron, and 0.1 part of manganese per million, besides a trace of hydrogen sulphide. Plate CXLIV shows the arrangements of parts, namely, wooden-slat aerator, roughing filter, final filter, and filtered-water reservoir. The capacity is 50 000 gal. per diem.

As the water is soft and contains manganese, it is treated best by thorough aeration and contact with gravel, followed by filtration through sand. Organic matter does not interfere, but the small amount of residue on evaporation (78 parts per million) and the presence of manganese are indicative of a water which precipitates its troublesome elements quite slowly. The filters are arranged to be drained daily so

*"Filtration of Public Water Supplies," p. 186.
†*Richmond Conservator*, Nov. 24th, 1904.
‡*Municipal Journal and Engineer*, 24, 520 (1908).



DEFERRIZATION PLANT

FOR

DR. HOMER GAGE,
Shrewsbury.

that oxygen may be condensed on the surfaces of the filtered material while out of service. The gravel filter is cleaned by flushing, the sand filter by scraping. The results have been satisfactory.

Demanganization.

Demanganization of water follows the same general laws as deferrization, but the speed of reaction is slower, one reason for which is that the mass of manganese usually present in ground-water is less than that of iron.*

At the Müggelsee works at Berlin, Director Anklam showed the writer how the red iron hydrate had been retained by the sand of the filter, while the precipitation of the manganese was delayed until the water had passed into the gravel of the under-drain, which gravel was coated with dark brown manganese hydrate. At Stargard (Pomerania), the water after passage through brick aerators was filtered upward through filters composed of 8 in. of coarse gravel, 11 in. of fine gravel, and again 8 in. of coarse gravel, arranged in superimposed layers. The water contains 2 parts of iron per million and small amounts of manganese. The iron is precipitated in the lower part of the filter, but Director Ehlert demonstrated that none of the dark brown manganese hydrate could be seen until the upper half of the fine gravel was reached. The cross-section of the filter showed: first, a red zone of iron hydrate; then one of clean gravel; then a brown, almost black, 4-in. zone of manganese hydrate; then another one of clean gravel. At Brunswick, Director von Feilitzsch showed the writer samples of sediment from different parts of the aerator and filter, which proved that in this case also the iron was precipitated earliest. At Hanover the writer was presented with three samples of precipitate as follows:

No. 298, from the distribution system before filters were introduced;
No. 299, from the filter material;
No. 300, from the water-tower.

These samples were analyzed, the results being shown in Table 18.

TABLE 18.

Number	298	299	300
Iron (Fe)	57.80%	34.72%	14.27%
Manganese (Mn)	0.30%	1.24%	7.15%
Ratio (Mn: Fe)	1:193	1:28	1:3

*Metallic hydrates precipitate from concentrated solutions faster than from dilute.

Sample No. 298 was taken from the pipes at a point where nearly pure iron oxide precipitated. The Hanover filters effect only a partial deferrization of the water, so that there is an after-precipitation in the water-tower. This after-precipitation is more indicative of the presence of manganese. Sample No. 298 is brownish red, No. 299, reddish brown, and No. 300, very dark brown, almost black, in color. The raw water contains 0.5 part of iron (Fe) per million and traces of manganese.

At Arad, Hungary,* Mr. Rudolph Hajek observed that water from the deep wells before and after deferrization was perfectly clear; also that there was no deposit in the pure-water reservoir at the deferrization plant after a year's use, but in the force main, from the pumping station to the distribution system, where the velocity of the water was only 0.85 ft. per sec., there were numerous brown flocs of manganese hydrate on the upper surface of the pipe and also a deposit upon the bottom. There was little deposit in the service pipes, but the galvanized-iron strainers of the water meters were found to have their holes almost completely choked with a yellowish-brown mass, although the strainers made of copper or brass were clogged but little. A pressure filter placed in the force main near the pumping station remedied the difficulty. The water contains only 25 parts of iron per million and traces of manganese. It has a hardness of 140 parts per million. Another interesting fact was noticed, namely, that the trouble due to manganese is greatest after periods of heavy rain.

Manganese has rarely given serious trouble in America. The first case known to the writer was, in 1898, when a well-water supply for a New England mill contained so much manganese that the well had to be abandoned. This case was mentioned by Mr. Hazen in a private communication, and is the earliest record of troublesome amounts of manganese in a water supply with which the writer is acquainted. In this case, the well was beneath a peaty layer overlying the water-bearing sand.

Manganese occurs in the waters of Reading, Mass., in some of the waters of Long Island, and in many other American water supplies, but usually in small amounts.

The Breslau Calamity.—In former years Breslau was supplied with filtered Oder River water. The Oder River becomes quite turbid at

Schillings Journal für Gasbeleuchtung und Wasserversorgung, Aug. 17th, 1907; *The Engineering Record*, 56, 517.

times, and the slow sand filters in use frequently passed a turbid effluent of questionable quality. The rapid increase of population in the industrial villages above the city caused a movement for the improvement of this supply. This movement was headed by the distinguished Professor Fluegge, of the Department of Hygiene of the University of Breslau, who, as is well known, is partial to ground-water supplies. This agitation led to a preliminary study, begun by Baurat A. Thiem in 1892, which resulted, before the close of 1904, in a ground-water supply from the Oder Valley above the city. This supply had a capacity of 16 000 000 gal. daily. It was collected by means of three hundred and thirteen 6-in. wells driven to depths varying from 33 to 40 ft. and placed 70 ft. apart. The wells are arranged in three lines and are overflowed at extreme high water.

The soil of the Oder Valley is made up of a layer of humus matter, silt, and clay, under which is a stratum of alluvial sand of various-sized grains. Beneath the sand is the diluvial clay 100 to 235 ft. thick, the top of which is from 30 to 55 ft. below the surface. The layer of loam, peat, and clay, is from 7 to 10 ft. thick. The ground-water level before beginning the daily pumping is from 20 to 40 in. below the surface of the ground. Beginning with January, 1905, ground-water exclusively was supplied to Breslau. By March 27th, 1906, the ground-water level had sunk to 25 ft. below the surface during pumping; previous to this date there had been little rain.

The deferrization plant consisted of a coke aerator and gravel filter, the plant resembling the Piefke system in design. The aerator is 2 m., the gravel filter 1 m. deep, and there is a small basin between aerator and filter.

On March 28th, 1906, the river rose and overflowed the third line of wells, and later, part of the second line. Soon the soil air caught beneath the flood water appeared in the wells, so much of it in fact that the air-pump could hardly keep the main free. At the same time the amount of iron increased to 440 and the manganese to 220 parts per million. For a time the deferrization plant removed the iron successfully, but the manganese passed on to the city and made the water useless for domestic purposes. As a result the ground-water supply was abandoned; filtered Oder River water was again supplied to the city, and three open filters were built to tide over the emergency.

Many explanations were at once offered, but even to-day no one explanation is universally accepted. One said that the manganese came from below the water-bearing strata, being forced upward by the pressure of the artesian water through from 100 to 235 ft. of clay and schist. Others, and among them, the geologists, engineers, and a majority of the chemists, believe that Luedecke has proved that the manganese was leached out of the humus-clay stratum by the overflowing river water. This position is hotly contested by the hygienists of the Breslau school, who affirm that the passage of flood water into the wells through the overlying clay stratum is a practical impossibility. Director Debussmann of the Water-Works made a thorough experimental study of this question and states that the river water can reach the wells in as short a time as twelve hours when the valley is overflowed.

American engineers who have used clay in dams and embankments will not place much faith in the impenetrability of from 7 to 10 ft. of river-bottom soil; and, although it may be true that some of the dissolved manganese diffuses from the upper stratum, it seems to be proved that river water did pass through it directly. At Reading the soil is not unlike that of the Oder River Valley, and the writer has repeatedly seen pools of rain water disappear into the ground during six hours of pumping.

Luedecke states that the Oder Valley soil contains much iron and manganese in the form of sulphides.* As long as these sulphides were not exposed to the air they remained unchanged. The lowering of the ground-water level brought air in contact with these insoluble sulphides, which then oxidized to sulphates and free sulphuric acid, which latter is an active solvent of iron and manganese. The acid could act as a solvent, because the soil is not rich in lime and other alkalies which would neutralize the acid. This process went on for a year and a half, when a sudden flood caused the appearance of the accumulated iron and manganese in the ground-water. Of course it is presumable that diffusion had something to do with this appearance, but this seems to be an inadequate explanation, because after a subsequent flood in September, 1906, the amount of iron in the water of the overflowed wells rose suddenly to more than 100 parts per million, and the water became acid to litmus. A measurement of the rise in ground-water

*"Gesundheit," 1907.

level showed that about 1 325 000 000 gal. of water had seeped into the water-bearing layer during this flood. This explanation agrees with the writer's experience at Reading, where the highest amounts of iron and manganese followed a flood or a sudden heavy shower preceded by a long period of comparatively dry weather. As before stated, there is more manganese in the Arad, Hungary, water supply immediately following a rain.

Several methods for removing the manganese from the Breslau water have been proposed. H. Luerig and A. Blasky* have advised the treatment of water after deferrization, first with lime, followed by aeration, sedimentation, and filtration, and finally with carbon dioxide gas—practically a softening process. Many of the wells, however, are unaffected by floods. They are in use at the present time, and, if the writer is informed correctly, new wells are to be sunk to take the place of those which, during floods, contain too much manganese to be removed by the present deferrization plant. In this way the ground-water supply is to be gradually restored, and the filtered river-water supply abandoned; the filters now being used for river water will ultimately be used for ground-water, as at Berlin.

Since writing the above there has appeared a paper by Professor Beyschaly, Director of the Prussian Geological Survey, and Dr. R. Michael, which gives an account of studies made to learn the cause of the trouble at Breslau.† They, also, believe that the calamity was due to the manganese leached from the top soil of the valley and not to the water from beneath the lower clay layer, which water contains little iron, only traces of manganese, and differs from the more superficial waters in nearly every way.

Design of Deferrization Plants.

The design of deferrization plants is governed largely by the character of the water. The subject is best discussed under the following heads, which are the names of the various parts of such a plant: Aerators; Subsiding basins; Roughing or contact filters; Final filters; Apparatus for chemical treatment.

Aerators.—If it is desirable to remove carbon dioxide from the water, the more thorough the aeration the better, and for this purpose

Chemiker-Zeitung, 31, 255.

†*Schillings Journal für Gasbeleuchtung und Wasserversorgung*, 51, 84.

one may use a great variety of devices. The tower type of aerator, such as is used in Germany, has the advantage of aiding the precipitation of the iron by contact with rough surfaces, which effect is not obtained in aerators consisting of simple sprays or superimposed perforated pans.

Where the water contains organic matter in colloidal solution and it is a disadvantage to liberate the carbon dioxide, it is often better to supply the requisite amount of oxygen by simple spraying or by admitting air into the suction of the pumps. The disadvantage of the tower type of aerator is that from 3 to 10 ft. of head is lost, and it seems to the writer that, if contact action is desired, it would be more economical to aerate by a simple spray, or superimposed perforated pans, or admission of air into the suction, and then pass the water through coarse gravel, thereby providing both aeration and contact without undue loss of head.

Subsiding Basins.—Where the precipitation is rapid, well-baffled subsiding basins are a decided economy, because they relieve the final filter of much work. In such cases they are to be preferred to a roughing filter of the same construction cost.

Roughing or Contact Filters.—These filters are simply beds of gravel through which the water passes either intermittently or continuously and either downward or upward. Experiments at Reading, as before noted, show how efficient these roughing filters are in preparing that water for final filtration. In most cases they are more economical than subsiding basins, and in many cases remove practically all the iron from the water. They should be designed, like some German filters, to deposit the iron all through the filtering layer and not on the surface only. The gravel should have rounded grains so that the bed may be flushed with ease.

The rate of filtration through roughing filters cannot be prescribed without knowledge of the water to be treated. For many waters a rate of 50 000 000 gal. per acre per day may be used. When higher rates are used, the retention of iron hydrate may be diminished. This factor also is dependent upon the character of the water.

Final Filters.—Final filters may be of varying degrees of fineness, ranging from the gravel which will be retained by a sieve having four meshes to the inch to sand having an effective size of 0.12 mm.

It is desirable when coarse material is used to cause the precipitation of the iron throughout the whole layer of filtering material instead

of near the surface, as is best in the case of surface-water purification. In Germany this is accomplished by grading the filter material. Filters of this kind, at Leipsic, Stargard, and Harburg, have been already described.

The aim of all deferrization plants is, of course, to increase the speed of precipitation, and it may be laid down as a general rule that the coarser final filters are most useful in waters which precipitate their iron rapidly and in coarse flocs. Waters which are difficult to treat, however, while they may be improved greatly by passing through a coarse roughing filter of ordinary thickness, almost invariably require a final filtration before being suitable for consumption. Mechanical filters are almost invariably used with waters requiring chemical treatment. There are no materials superior to sand and gravel for use in filters.

The properly treated water may be filtered through the final filters at almost any practicable rate, and for this purpose mechanical filters are to be preferred as a rule. However, slow filters at 15 000 000-gal. rates may be found more economical in many cases, especially where preceded by roughing filters, which latter would increase the quantity of water filtered between scrapings. At Far Rockaway, N. Y., a growth of algæ forms a mat on the surface of the bed, which mat can be easily rolled and removed, taking with it the accumulations of iron at the surface of the bed, and reducing the cost of scraping.

Apparatus for Chemical Treatment.—The apparatus for applying sulphate of alumina should be the same as that used for the purification of muddy river waters. Lime can be added either in the form of milk of lime, or, more economically, as a saturated solution, as is practiced at St. Louis, Mo., and other cities. Clay is best added in the form of a suspension. The clay for this purpose should be selected carefully, and that which has the largest percentage of finely divided particles should be chosen for use. It should be mixed with water, the heavier particles should be allowed to subside, and the suspension of fine particles should be fed into the water by an orifice tank through which the suspension is circulating continuously. A thorough mixing of the chemicals with the water to be treated hastens the speed of precipitation. Very often it is desirable to subject waters treated with chemicals to a preliminary filtration through a roughing filter before giving them a final filtration. The decision in this case

should depend upon the comparative costs of coagulating basins and roughing filters of like efficiencies, and the costs of scraping or washing the final filter with and without such preliminary treatment.

The writer feels disinclined to discuss the arrangement and choice of ozone apparatus with the limited data at hand. It is enough to say that the ozone treatment is simply aeration with ozonized air, and the ozonization of the water takes place under similar conditions, but with the exclusion of the external air.

Conclusion.

The first requisite for the deferrization or demanganization of a water is a thorough knowledge of the water itself, not only from a chemical, but also from a physical and geological standpoint. All alkaline ground-waters will precipitate their iron ultimately, although it may require months in some cases. Those moderately hard waters free from any considerable amount of organic matter, which begin to throw down iron as soon as they appear above the surface of the ground, need only to be aerated and filtered through coarse material; while those which precipitate iron more slowly require more aeration and more contact with filtering material and the accumulated ferric hydrate to get good results.

Those waters containing interfering amounts of organic matter must be specially treated. Either there is an excess of organic matter holding the iron hydrate in suspension, an excess of iron holding the organic matter in suspension, or the organic matter is in colloidal solution and interferes with the precipitation of the ferric hydrate.

In the first two cases it is necessary, in the language of physical chemists, to "balance" the electric charges of the suspended particles. That is, if the iron be in excess, add a negative colloid like suspended clay or organic matter; or if organic matter be in excess, add iron or aluminum hydrate. If organic matter in solution interferes, it is possible to add lime, which forms an insoluble compound with the humus matter; to treat the water with ozone which destroys it, or to increase the bulk of precipitate by adding both positive and negative colloids, for example, sulphate of alumina and clay, and, by the absorptive action of the greater bulk of precipitate, to overcome its effect.

In a few words, the aim of every process should be to allow time

enough for the oxidation and coagulation of the iron or to shorten the required time by aeration, by contact, by balancing the several kinds of suspended colloidal matter so as to mutually precipitate one another, or, if necessary, by increasing the mass of precipitate so as to outweigh the effect of all interfering substances which may be present in the water treated.

The writer wishes to acknowledge the assistance and advice of many engineers and chemists, most of whom have been mentioned. He is especially indebted to the Reading Water Board and its officers for the opportunity to test various arrangements of apparatus on a large scale.

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PAPERS AND DISCUSSIONS

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THE OPERATION OF PASSENGER ELEVATORS.

By Reginald Pelham Bolton, M. Am. Soc. C. E.

To be Presented February 17th, 1909.

The subject of the conveyance of passengers to and from the floors of high buildings is one in which the public of New York City, by reason of the city's peculiar physical conditions, has a particular concern, and in which there is much of interest for residents in other cities, as well as for members of the Engineering profession.

The great increase in the number of high buildings in the metropolis within the last decade is evident to any casual observer, more than 500 buildings exceeding 125 ft. in height having been erected within a period of less than 20 years. In these, and in other buildings of less height, many thousands of elevators are in operation, carrying a traffic far exceeding in numbers the passengers on all the railroads of the Greater City, and reaching a very large total distance of daily travel.

Between June 1st, 1903, and January 1st, 1908, there were erected, in the down-town district, south of Chambers Street, office buildings having a total occupied area of 3 157 596 sq. ft., and affording, on an average occupancy of 1 person to 130 sq. ft. of rentable area, space for a business population of 24 000 persons. Nearly all this space in

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

business and office buildings is at an average height exceeding 13 floors above the ground, and it may be estimated that not less than 188 elevators are required to accommodate this tenancy.

In the lower part of the Borough of Manhattan there are now twenty-six buildings which exceed 17 stories in height, and their occupied or rentable area above the ground floor amounts to about 5 044 630 sq. ft. There are 266 elevators in these twenty-six buildings, and their average travel is 4 400 miles per 8-hour day, their passenger-carrying capacity being 2 896 000 persons in 8 hours.

The Department of Buildings reports the existence, in the Borough of Manhattan alone, of 9 000 passenger elevators and 12 000 freight machines. During the past six years more than 4 000 elevators have been installed in this area; and between 1903 and 1907, about 9 725 921 sq. ft. of rented area has been added in business buildings, 10 965 837 sq. ft. in lofts and stores, and 3 178 248 sq. ft. in factories, a total of 23 870 006 sq. ft., or about 550 acres.

In the business buildings alone this may be assumed to provide accommodation for about 65 000 persons, a rate of increase of about 13 000 persons per year, who depend upon elevators for access to and from their places of business.

Such an important adjunct to the operations of modern business as the passenger elevator has thus become, connected so directly with the comfort and convenience of the public, as well as the prosperity and value of immense property interests, deserves a closer degree of attention than has been hitherto bestowed upon its operation, and some study of the conditions which in this form of transportation have proven so successful, as compared with the failure of transportation methods in the horizontal direction, will not be unworthy of consideration by the Society.

Elevators, like railroad trains, are appliances which operate in combination with the human element making use of the mechanical features.

The reciprocation of elevators and of trains, if they should be conducted without reference to passengers, might afford a maximum amount of operation with a minimum of utility. Their movements within any given period of time are reduced and limited by the time introduced into their operation, by passengers entering, riding in, and leaving the appliance.

It will be seen, as the study of the operating conditions of elevators proceeds, that they are subject to much the same limitations as in the case of the operation of railroad trains and surface cars. A large building equipped with a bank of elevators, operated on a schedule system, is practically a short-distance railroad, having stations only a few feet apart. The mere fact of the travel being vertical, has little bearing on the comparison, as it is simply a question of power operating in one direction adversely to gravity, while the force of gravity is utilized in the reverse direction.

The limitations before referred to are less connected with the designed speed of the apparatus than with the time occupied by the human freight in utilizing the appliance. As in surface transportation, it is not the alert and active passengers, but the slow and stupid ones who bring about the delays, aggravated in both forms of transit by ill-designed means for admitting of the passage of these persons to and from the vehicle. As one obstructive person can delay the movement of a train from a station, and thereby hinder all trains following on the same track, so the actions of one person in one elevator will affect the operation of others running on a "schedule," notwithstanding the fact that each elevator possesses its own independent line of travel.

Where too large a traffic is imposed upon an elevator or set of elevators, a delay will result, not only in time occupied in handling those persons, but in an additional loss of time due to crowding and pushing past one another. This is exactly what is occurring to-day in surface transit, and, as a result, the designed operation of trains or elevators is modified, and to a great extent fixed, by the conditions set up by the passengers.

An attempt to carry in any elevator an undue number of persons per trip brings about delay and less satisfactory service, and results in a less number of persons being carried per hour. A maximum of convenience and efficiency is attained, when, with intelligently designed means of ingress and egress from the cars, an average number of persons is carried per trip each way, bearing the relation to the floors which are served, of four persons to ten floors.

The passenger elevator is an appliance having designed limitations of motion, and certain capacities for lifting and lowering live loads in its cars, up to a maximum, at predefined speeds. The variation in-

troduced into its operation by varying loads is not only a modification of traveling speed, but of the time which is occupied in the acceleration of the car from a state of rest to that speed, and of its retardation from that speed to a state of rest.

These modifications also vary in the two directions of travel, for on the up-run the force of gravity is adverse to the load and to its acceleration, but is in favor of its retardation, while gravity acts in the reverse way regarding each upon the down-run of the car. The effects of gravity may be modified by balancing or counterweighting, but the modification merely distributes the effects between the live load and the counterweight, and cannot usefully be extended much beyond the average live load.

Very considerable variations, therefore, may exist between different elevators, not only in the matter of speed with different loads, but in capabilities of acceleration and retardation at various loads; and the efforts of designers have been very properly directed toward bringing about the most economical as well as effective combination of lifting power, speeds, and retardation over the range of loads from full capacity of the car, to that of a car containing only the operator.

As the operations of starting and stopping are necessarily exercised at very frequent intervals, the capabilities of the apparatus in these functions may be of greater importance than running speed. Thus a machine having a certain car-speed with a corresponding live load, is exercising that particular speed only over such part of the distance of the travel of its car as remains over and above the distances occupied in the process of acceleration and retardation.

The point may be illustrated thus: A car serving 15 floors above the ground floor, travels, say, 384 ft., making 7 stops each way, or 16 separate flights. It has the ability to accelerate and retard its average speed in 10 ft., occupying a period in each such operation of 2.4 sec. It will thus occupy $2.4 \times 16 = 38.4$ sec., and the distance covered in this period, is $10 \times 16 = 160$ ft. The remainder of the distance, or 224 ft., is covered (at a designed traveling speed of 500 ft. per min.) in 26.9 sec. If the traveling speed were 600 ft. per min., with similar acceleration and retardation, the only gain would be $4\frac{1}{2}$ sec. on the whole trip, so that, to afford any substantial advantage, the faster machines must also decrease the time of acceleration and retardation. Mere increase of running speed, where stops are frequent

and flights are correspondingly short, does not show commensurate advantage.

Since the functions of acceleration and retardation thus occupy so important a part of the operation of any elevator, it is evident that operating conditions which bring about the increase of the number of such functions, and the consequent reduction of the distance in which the real speed of the appliance can be exercised, are of great importance.

If an elevator be charged with traffic so that its operation involves a stoppage at every floor, which in business buildings may be taken to be commonly arranged at a distance of 12 ft. apart, then there is no time in which, with an average distance of acceleration and retardation of, say, 15 ft., its designed speed would come into play. A machine having a speed of 8.33 ft. per sec., would, in point of fact, develop only an average speed of 3.73 ft. per sec.; but if the traffic were related to the elevator in such a way (or rather, if the number of elevators were so adequate for the traffic) as to require stops at intervals or flights of two floors, or say, 24 ft., the mean speed would rise to 5.13 ft. per sec., and the gain in the round-trip time in a 16-story building would be 28 sec., a substantial saving of nearly half a minute.

Fig. 1 shows the curves of mean speeds, in feet per second, resulting from functional distances of 10 and 15 ft., and a traveling speed of 500 ft. per min., at various intervals or flights, up to 30 ft.

The speed of a passenger elevator is thus seen to be closely related to the traffic which is imposed upon it. If that traffic be such as to increase the number of stops, and so to reduce the average flights of the car below a distance required for corresponding acceleration and retardation, then the speed capability of the machine is not exercised at all, and the mean speed is reduced in proportion to the increase of traffic.

In a recent book on elevator service by the writer, the statement is made that when carrying more than a certain number of persons per trip an elevator would actually fall off in traffic. This result can be readily comprehended when it is realized that excess of load involves, not only a decrease of traveling speed, but also an increase of the distances required for acceleration and retardation, all involving losses of time.

The loads, traveling speeds and stopping distances, observed by

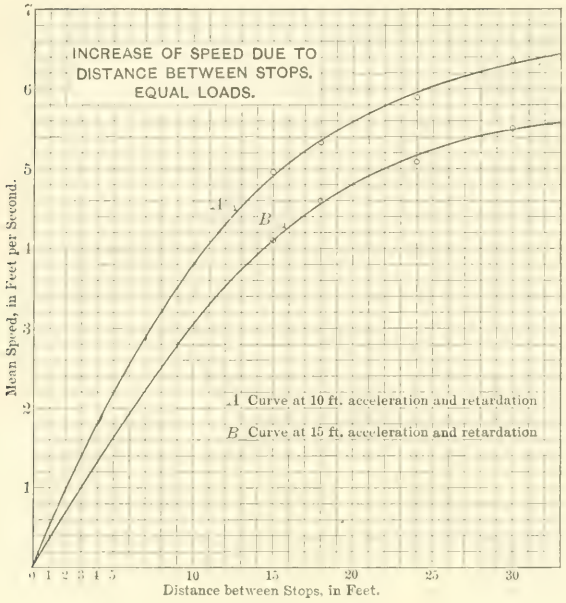
INCREASE OF SPEED DUE TO
DISTANCE BETWEEN STOPS.
EQUAL LOADS.
Mean Speed, in Feet per Second.
A
B
A Curve at 10 ft. acceleration and retardation
B Curve at 15 ft. acceleration and retardation
0 1 2 3 4 5
10
15
20
25
30
Distance between Stops, in Feet.

FIG. 1.

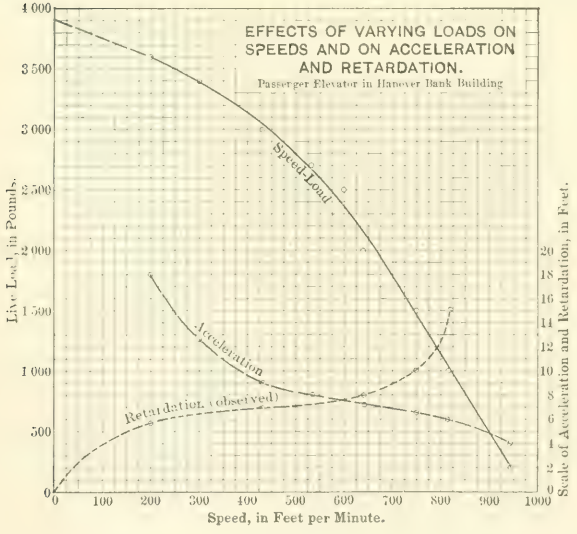
EFFECTS OF VARYING LOADS ON
SPEEDS AND ON ACCELERATION
AND RETARDATION.
Passenger Elevator in Hanover Bank Building
Live Load, in Pounds.
4 000
3 500
3 000
2 500
2 000
1 500
1 000
500
0
Speed-Load
Acceleration
Retardation (observed)
0 100 200 300 400 500 600 700 800 900 1000
Speed, in Feet per Minute.
Scale of Acceleration and Retardation, in Feet.
20
18
16
14
12
10
8
6
4
2
0

FIG. 2.

J. R. Furman, M. Am. Soc. C. E., on the powerful hydraulic elevators installed in the Hanover Bank Building, in the City of New York, afford an excellent and authentic instance of the variations introduced into the operation by increasing loads, and are plotted in Fig. 2, together with the curves of distances for acceleration and retardation, from a live load of 180 lb. to one of 3 600 lb., or say, from the weight of the operator alone, up to a weight corresponding to about 25 passengers and the operator.

Applying the capacities of such a machine to the service of twenty-five floors above the ground floor, the mean speed between stops will be varied by the load and the distance between stops required by that load, as shown in Table 1.

TABLE 1.

Load, in pounds	1 000.	1 500.	2 000.	2 500.	2 700.	3 000.	3 400.	3 060.
Corresponding passengers	7.	10.7	14.	17.	19.2	21.4	24.2	25.7
Relation of passengers to floors served	0.28	0.42	0.56	0.72	0.77	0.85	0.97	1.02
Corresponding flights, in feet	43.	30.	21.5	17.	16.	14.	12.	12.
Average speed between stops	653.	574.	454.	352.	325.	285.	210.	175.

These results give a curve of passengers per hour, plotted in Fig. 3, corresponding to passengers per trip. The hourly traffic carried increases rapidly, until the point is reached where the relation of passengers to floors served is as 4 to 10, then it increases at a reduced rate, until the relation of 6 to 10 is reached, and then it decreases.

When a relation of one person to each floor is reached, the traffic per hour is no more than that when about 3.5 persons per trip are carried.

These figures also show that the 4 to 10 traffic is 94% of the maximum attainable, and that this amount is carried in 60% of the time of the maximum.

The difference in favor of the 4 to 10 relation is even more marked when one-way hourly traffic is computed, as in the small upper curve, which shows that when running on the 4 to 10 basis all one way, very nearly the maximum travel is attained, and that an elevator to which had been apportioned a greater amount of traffic, would fall off very considerably in hourly traffic capacity.

The developments of elevator travel have brought about a general

demand for a convenience in point of time which is represented by an effective service within a period of about 3 min. for a round trip; therefore, it becomes very desirable to determine the amount of traffic which shall be apportioned to an elevator, and thus to learn what imposition of live load and corresponding limitations of flight are to be introduced into its duty.

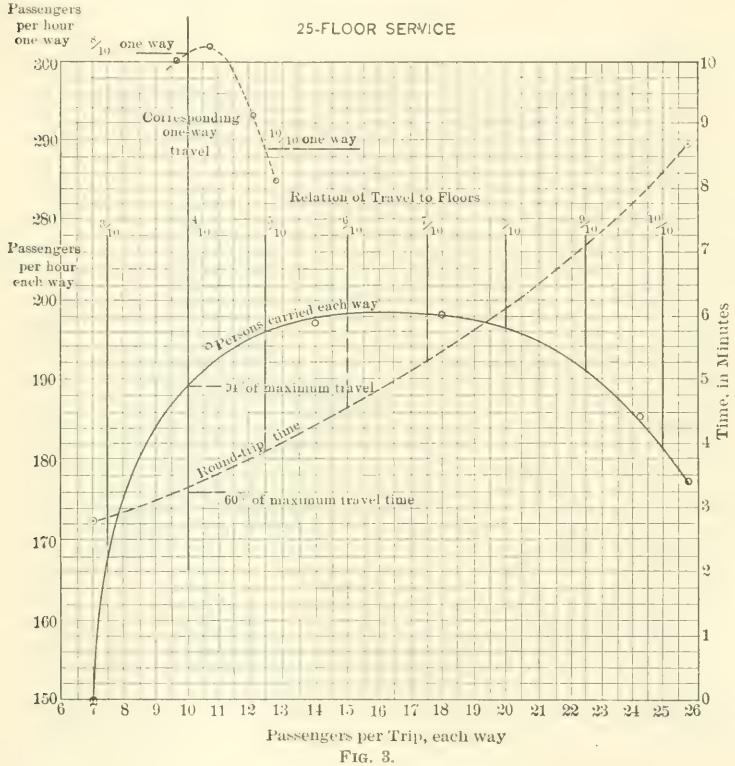

FIG. 3.

In the past, the former of these two elements has been rather loosely assumed by designers to be represented by an average of from 40 to 50% of the car capacity of an elevator. The lack of definiteness in this assumption is apparent. Cars may be and are of all sizes, due to eccentric conditions of the building, or to the personal views of architects, their capacity for passengers bearing no relation to the developed traffic. A large car designed to operate under conditions of 40% of its full load may be allotted traffic averaging 80% or

20%, involving a totally different condition from that for which it was destined.

As to the time occupied in passenger movement in the foregoing computations, and in Fig. 3, the observations published elsewhere by the writer indicate that the average passenger occupies for his or her entrance to the car at the starting point, exit therefrom, re-entrance, and final exit, a total period of 12 sec., and in addition, to every round trip, 9 sec. should be added to cover gate opening and start, at the top and bottom of the run.

The combination of time thus established, covers the probable requirement of a landing to each passenger, and this is a duty which the machine must be prepared and proportioned to perform. It is not desirable to assume that any pre-determined proportion of passengers share a landing with others, since the capacity of the elevator must be such as will provide for the contingency of a separate landing for each.

The time thus established for passenger movement appears to be in very close agreement with actual conditions. Some trials in which the time occupied in stops or landings are given, together with the time of round trips, are plotted against the outline of a calculated or average trip in Fig. 4. One is that of the Hanover Bank elevators, already referred to, and the other, that of a plunger machine in the Trinity Building. It will be seen that the agreement of the actual with the computed conditions is very close.

A series of most interesting observations has been undertaken recently at the writer's suggestion, by Mr. Ernest M. Hill, on the traffic of the Ellicott Square Building, Buffalo, on August 21st, 1908. These observations are confirmatory of the assertions herein made, as to the time occupied by passenger movement. This building is provided with two public entrances, at each of which are located two banks of Otis hydraulic-cylinder elevators, speeded to an average of about 350 ft. per min. Of these, five were in operation at the Main Street entrance, which is the busier of the two, and on these, a record was taken of every person carried either up or down, on every trip, of each elevator, during a period of ten hours, which averaged as follows:

Up, per car per trip	4.93
Down, per car per trip	4.53
Mean, per car per trip	4.7

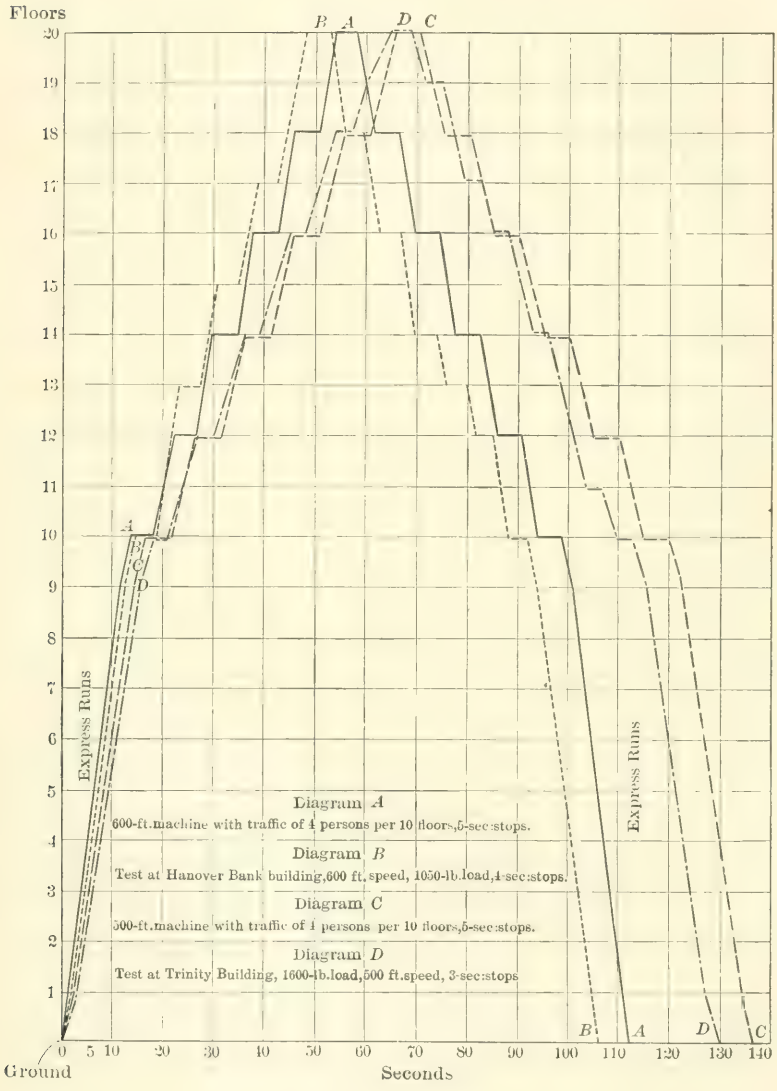
Floors
Ground
Seconds
Express Runs
Express Runs
Diagram A
600-ft.machine with traffic of 4 persons per 10 floors,5-sec:stops.
Diagram B
Test at Hanover Bank building,600 ft. speed, 1050-lb.load,4-sec:stops.
Diagram C
500-ft.machine with traffic of 4 persons per 10 floors,5-sec:stops.
Diagram D
Test at Trinity Building, 1600-lb.load,500 ft.speed, 3-sec:stops

FIG. 4.

The round-trip time of these five cars averaged 2.09 min., which closely agrees with a calculated time of 2.01 min. During the busiest hour the traffic increased to 5.52 per trip, when the round-trip time lengthened out to 2.16 min. This is the same as the computed time based on the allowances of time, as herein stated, to each passenger.

These results, therefore, confirm the time allowance of 12 sec. to each passenger, and show that such a traffic reduced the speed between landings to 260 ft. per min.

The passenger elevator is thus seen to be an appliance in which, whatever be its designed functions, the time of its operation is subject to large modification by the passengers making use of it. It does not, therefore, stand in the same order as other machinery, but its operation is to be considered as in part that of the human beings carried, both as to numbers, as to their individual actions, and last and not least, in part that due to the idiosyncrasies of the operator.

Herein is to be found the reason for the wide variation in round-trip times, and in resulting convenience or its opposite, of machines of similar character and power, and much explanation may be derived of the difficulties under which designers of elevators have labored, in ascertaining the various elements which go to make up the speed capabilities of their apparatus.

The desideratum is evidently the attainment of maximum powers of acceleration and of retardation, and of the maintenance of traveling speed, with varying loads.

The ideal machine, for the attainment of maximum results, in traffic, will be that which will possess the power to attain with all loads, up to the desirable maximum, the designed speed in either direction, and in combination with a system of counterbalance related to its real average live load, will with all loads occupy the same distance for combined acceleration and retardation in either direction.

In the past, too great attention has been paid to the subject of operating economy, and that attention may now be profitably diverted to the more important question which determines the real work and economic value of the apparatus, namely, the amount of passenger traffic which the elevator shall be apportioned to convey.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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WATER-WORKS VALUATION AND FAIR RATES, IN THE LIGHT OF THE MAINE SUPREME COURT DECISIONS IN THE WATERVILLE AND BRUNSWICK CASES.

Discussion.*

By Messrs. C. S. Burns, James Nisbet Hazlehurst, and Allen Hazen.

C. S. Burns, M. Am. Soc. C. E. (by letter).—This paper will be read with interest, and those engaged in valuation work will be grateful for the added light thrown upon a subject which has been enveloped in ambiguity. Doubtless no subject of equal importance has caused more diversity of opinion among experts than this question of valuation of quasi-public utilities. The Brunswick opinion, cited by the author, is especially interesting, as it represents the amended ideas of an eminent judge after further consideration of a subject previously studied, presumably with considerable care. Even in the Maine decisions, however, some rules are prescribed which upon close analysis appear to lead to ambiguity and to conflicting results relative to franchise value. Mr. Burns.

It seems clear that franchise value cannot be considered in determining reasonableness of rates, because the franchise has no value, other than the opportunity that it creates for the safe investment of capital, and unless the capital thus invested produces returns greater

* This discussion (of the paper by Leonard Metcalf, M. Am. Soc. C. E., printed in *Proceedings* for October, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Burns.

than the market rate fixed by the law of supply and demand, then the franchise has no value, while, on the other hand, if the rates are higher than fixed by the law of supply and demand regulating the investment of capital in similar undertakings, then they are unreasonable by just that much. In other words, franchise value is the measure of the unreasonableness of the rates, and neither can exist unless the other is present. Moreover, it requires an unusual stretch of the imagination to conceive of a condition of affairs that could justify a city or town in paying for a franchise to occupy its own streets simply because that privilege had been previously extended to a corporation. If the municipality reimburses the corporation to the extent of what the property is actually worth to the city, including a fair estimate of the expense that the city would have to incur to develop the business, there can be no reasonable grounds for complaint.

To state that "the reasonableness of the rates relates to both the owner and the customer, but in case of conflict the rates must in any event be reasonable to the customer", followed by the stipulation, "A public service company cannot lawfully charge more than the services are reasonably worth to the public as individuals, even if charges so limited would fail to produce a fair return to the owner upon his property or investment," is unquestionably correct. If pursued to its logical conclusion, this theory would seem to require the consideration of evidence to show what the cost would be to the consumer, if permitted to serve himself by the substitution of another equally efficient, or different, structure to do the same work; and, in the light of the further statement, that "Communities are entitled to the benefit of natural advantages, and the company is not entitled to charge enhanced rates based upon the cost of using the more expensive source," it is difficult to understand why the testimony covering this point in the Portland case was not given consideration in conformity with the above rulings.

The original cost is frequently brought forward and unduly emphasized by corporation experts, with the plea that the valuation of the property should not be less than it cost the company, less proper allowance for depreciation.

However, the original cost should have no more consideration, in the case of a company having built works under difficulties and at a time of abnormal prices of material and labor, than it should under the reverse condition, or in case of a company having purchased a property at foreclosure sale at a small fraction of its structure value. If a property having a structure value of $100 000 were purchased by a private company for $25 000, it would manifestly be unjust for the municipality to take it at a later date under an appraisement based upon the original cost of $25 000, thereby taking advantage of the company's good fortune in having been able to acquire the property at

a bargain at a time when, perhaps, the municipality could not have purchased the works so advantageously, if at all. On the other hand, it is equally unjust to require the municipality to pay an exorbitant price for a property based on an original cost loaded with promotors' fees, excessive bond discounts, or unreasonable construction cost of any nature, whether it be due to high prices of material, poor management, lack of engineering investigation, or what not. Mr. Burns.

If a company undertakes the construction of a water-works system when prices are abnormally high, and is willing to finance the project with such disadvantageous conditions, under a franchise granting the municipality the option of purchase by appraisement, it does so of its own free will, and should not complain subsequently if its property be appraised at market prices differing from the original cost.

It is true that the "reproduction cost" has always been recognized as pertinent and important, though not controlling, in valuation cases. Why should it not be controlling in determining the price that should be paid for the property in case of purchase by the municipality?

Reasonableness of rates could not well be determined by this procedure, because there would result a continuously changing system of rates, which would be impractical of application, and, therefore, the only practical basis of valuation for rate-fixing purposes should be the average prices extending through a period of years, or what might be termed the normal prices. Normal prices appear to be the proper basis of valuation to be used in fixing rates, but not so in the purchase of the property.

When a municipality decides to exercise its prerogative, and purchase a water-works system, as provided under the terms of the franchise, it is entirely practical to appraise the works on the basis of current prices, and it seems proper to compute the reproduction cost on that basis, and use that result with proper allowance for depreciation (both physical and functional), as the correct valuation of the plant. Whether the result thus found is greater or less than what it would have been a year previous, or what it might be at some future time, is a mere incident, and should have no influence in the case. If the municipality elects to purchase the property at the present time, it should pay the present value, whether that be greater or less than the normal or average value. In fact, it can make but little difference either to the purchaser or the seller whether an appraisement be made at high prices or low prices, providing the prices used are current and up to date, but to use prices of a year or two ago may be of the greatest injustice to one of the parties. It would be interesting to know what special arguments were offered *pro* and *con*, to cause the ruling of the Maine Courts fixing the assumption of prices prevailing at a time long enough prior to the date of taking to cover the probable period of construction.

Mr. Burns. It is universally true that, when prices are abnormally low, rates of interest are high, and *vice versa;* the law of economics is an equalizing force affecting both the seller and the purchaser. By way of illustration —using the Portland case cited by the author—suppose the price of cast-iron pipe had been taken at the rate prevailing at the date of taking. The reproduction cost would have been $80 000 less than it was found to be under the Maine ruling requiring the price to be assumed at the market rates of two or three years prior.

The author does not present data to show the relative rates of interest that the city would have had to pay upon the bonded debt at the two periods in question, but the financial conditions throughout the country have been such that it is safe to predict that the municipality should have been given the benefit of the reduced price of material; and even after making the $80 000 reduction, the actual annual interest paid by the municipality on the bonds issued (assuming the plant to have been bought with the proceeds of a bond isue) would have exceeded the interest paid on the larger bond issue two years prior.

Again, considered from the standpoint of the seller, there seems to be no reason in equity why the plant should be valued upon prices other than those in vogue at the immediate date of sale. If prices are low, he has the opportunity to reinvest his capital in similar enterprises at those very same prices, hence, no injustice has been done by using the current prices. Why should the valuation be based on prices of a year or two prior, when the opportunity for reinvestment at those prices has long since gone by? Suppose the city, in purchasing pipe two years ago at $29 per ton, ordered a surplus of pipe, and stored this surplus pipe in the warehouse. Certainly no one would argue that this stock on hand is worth $4 per ton more than new pipe, and yet such an argument would be just as sound as to say that the pipe in the ground is worth $29 per ton now simply because the municipality would have had to pay that price for it two years ago.

There can be no rule or guide to the appraiser more fair than the principle quoted by the author: "The worth of the service to the consumer." This rule, intelligently applied, should be applicable in determining "fair rates" as well as present value for transfer of the property.

It is to be hoped that discussion will follow concerning functional depreciation, covering some of the customarily disputed points. Water companies have frequently employed able engineering advisers who have produced every possible argument to swell the value of the property, while the municipalities have as frequently failed to appreciate the necessity or value of such assistance, leaving the matter in the hands of some local attorney who finds himself overwhelmed with evidence beyond his comprehension, and is unable to meet the argu-

ments. The result has been that many unsound arguments have passed unchallenged, and methods of calculating present values wholly favorable to the corporation interest have prevailed so frequently as to have become classic and accepted without question or serious combat. Mr. Burns.

An example of this condition of affairs is illustrated by a recent case where the water company's expert maintained that a proper item to be considered in the cost of reproduction is the presence of a street railway located directly over the pipe line. He argued that the extra cost of taking up and relaying the track should be added as an item of value, on the theory that such expense would necessarily be incurred in duplicating the system. It is needless to say that the city's adviser easily combatted the argument by the mere statement that the pipe line need not be located under the track, is worth less by virtue of such location, and would not be thus located were the city to build its own plant.

But what about street paving? Appraisers have quite uniformly agreed upon the fairness of allowing extra value to a pipe lying beneath a paved street. But why do so, unless it is necessary for the pipe to be thus located? If the franchise provides that the paving over the pipe shall be laid and maintained at the expense of the water company, then it would be fair and equitable to allow extra for the paving; but, in the absence of such a provision, there can be no equity in allowing extra therefor, thus requiring the municipality to pay a foreign company a double premium upon an item for which the city itself has already paid.

It seems to be more logical to treat street paving as a cause for functional depreciation, for if a pipe line lies under a paving, it certainly costs more to maintain it, every street connection that requires repairs is more inaccessible, and, in fact, the street paving is decidedly an objection to the water-works system. If the pipe line could as well be laid between the sidewalk and the curb line, as is usually the case in residence districts, or in the alleys in commercial districts, thereby avoiding the paved streets, then the paving should be treated as an item of depreciation rather than contrariwise.

If an investor were purchasing a water-works system, it is doubtful if he would be inclined to pay any more on account of the pipes being covered with an expensive street paving; indeed, if the franchise required the water company to take up and replace all paving disturbed in connection with excavations for repairs, the prospective purchaser would insist upon a smaller price on account of the street paving.

Regarding physical depreciation, there can be no more fair method of calculation than to compute the annual cost of a new article, and ascertain what value of the old article will give the same annual cost to the user. By long established custom, interest is payable annually, taxes are collected annually, and the year is a convenient unit of time to use in the calculation.

Mr. Burns. Accepting the above hypothesis, the present value of any article, or machine, may be defined as that sum which will make its annual cost to the taxpayer, or user, the same as the annual cost of a new machine capable of doing the same work. The annual cost of any machine is the interest on its capital cost, plus the annual charge for renewal fund, plus the cost of maintenance and operation (if any).

The renewal fund is that sum which, placed at compound interest annually, will amount to a total sum at the end of the life of the machine sufficient to replace it. As a simple illustration, take a standpipe twenty years old, the total life of which is estimated to be thirty years, the cost of maintenance and operation being the same for an old structure as for a new one, assuming no functional depreciation to have taken place due to the growth of the town or changed local requirements. Assume the cost of a new structure to be $10 000. Then the annual cost of a new structure is as follows:

Interest on the capital investment at, say, 5%	$500.00
Annual renewal fund invested at, say, 3% = 2.1019%	210.19
Annual cost of maintenance (painting, etc.) = K.	
Total annual cost	K + $710.19

For a similar structure twenty years old, having ten years yet to serve, its present value being the unknown quantity, x, the annual cost is as follows:

Interest on capital investment, 5%	$0.050000 x
Annual renewal fund (10 years life, 3% basis)	0.087231 x
Annual cost of maintenance, as before, = K.	
Total annual cost	K + $0.137231 x

Hence $K + 0.137231\ x = 710.19 + K$, from which x, the present value, equals 5175.14 or $51\frac{3}{4}\%$ of the cost of a new structure.

The present value in percentage of the cost of a new structure could as well be determined directly by the ratio $\frac{5 + 2.1019}{5 + 8.7231} = 51\frac{3}{4}\%$, based on the author's sinking-fund data, Table 1. These results can be computed rapidly from sinking-fund tables with the aid of the slide-rule.

To facilitate calculations of this nature, the writer has constructed nine diagrams by the above method, and from these the present value may be read directly as a percentage of the cost of duplication. One of these diagrams is reproduced herein as Fig. 1. These results will differ slightly from the author's tables, as the renewal fund has been assumed to be invested at the beginning of each year instead of at the year's end. The reason for this is that most statutes provide for the

Mr. Burns.

INTEREST AT FOUR PER CENT ON CONSTRUCTION DEBT.
RENEWAL FUND INVESTED AT TWO PER CENT PER ANNUM AT
BEGINING OF EACH YEAR.

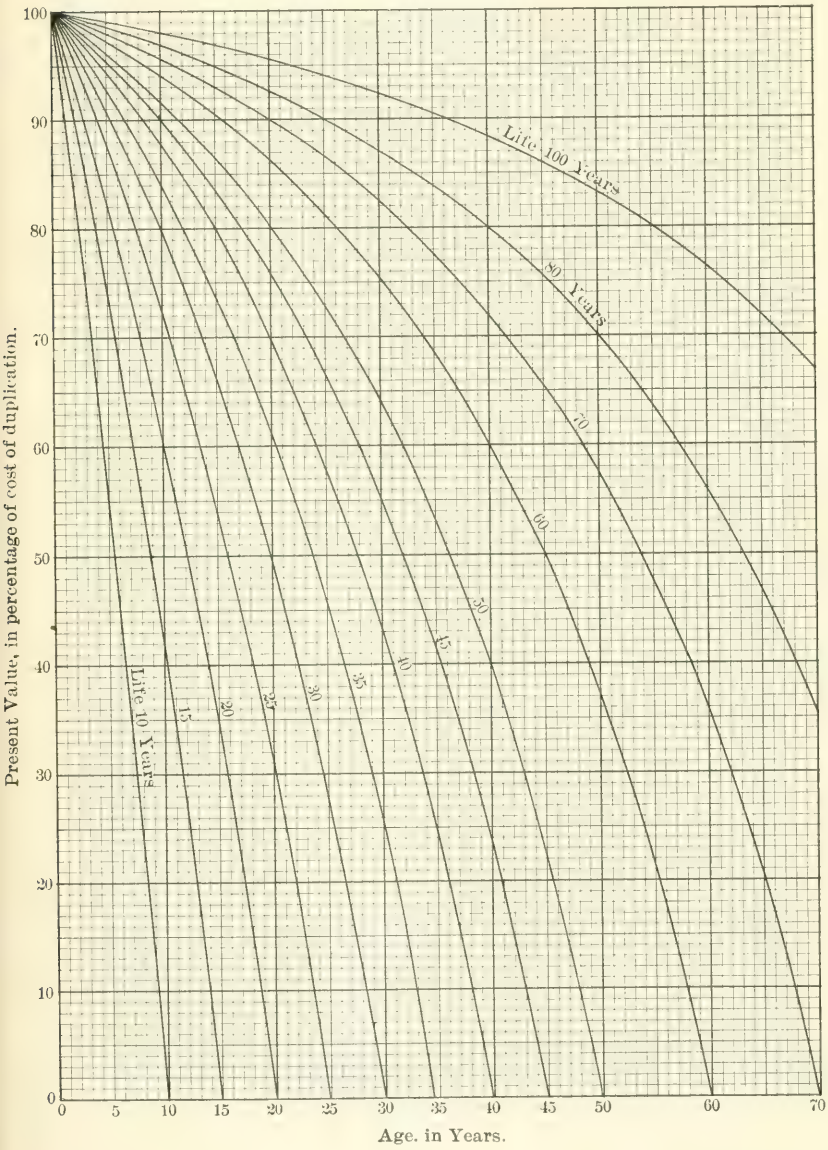

FIG. 1.

Mr. Burns. collection of taxes at the beginning of the year, and the renewal fund is nearly always collected in time to be invested at least as soon as the structure is completed and put into service.

The diagrams have been constructed for 2, 2½ and 3% for renewal fund investments, and for 4, 5 and 6% interest on capital investment. Before these diagrams were prepared, in 1905, letters were sent to fifteen of the leading banks and trust companies of Kansas City, St. Louis, Chicago, and New York, to ascertain what rate of interest each would agree to pay upon a sinking fund, contributed at the beginning of each year for twenty years, or more, and the replies ranged from 2½ to 3%, the fund being protected by surety bond at the expense of the bank in each instance.

If the article for which the present value is sought is a machine the cost of operation of which is dependent on its physical condition, this factor must be introduced into the calculation, and correction must be made therefor before applying the diagrams.

It may appear to some that this discussion is unduly partial to the municipality. It is not thus intended, although it must be apparent, to those who have followed the subject closely, that the corporation interests have developed their side of the subject far more skilfully than have the municipalities, due to the short-sighted policy of so many city officials in failing to employ competent experts to aid their legal advisers. Court decisions are based upon evidence, and influenced by argument, consequently it is but natural that the greater number of cases have brought forth decisions favorable to the party best prepared for argument.

Mr. Hazlehurst. JAMES NISBET HAZLEHURST, M. AM. SOC. C. E. (by letter).—The writer is particularly interested in the sub-topic "Monopoly Value of Land and Water Rights."*

As has been pointed out, the term "Monopoly Value" is loosely applied in such cases, since, from the franchise, the power, right, or privilege to sell water as a commodity may or may not be exclusive, depending on the precise terms of the grant, and, if exclusive, its duration in time may clearly affect such monopolistic value. While the Courts have generally held that the exclusive right to sell water to a city by individuals or corporations must not have for its object the creation of a monopoly, and cannot be enforced, yet certain cases may be recited showing a shadowy line of demarcation, as for instance, the case of Oakes *v.* Cattaraugus Water Company (N. Y.), 38 N. E. Rys., 461, wherein an agreement to refrain from forming a corporation for the construction of water-works and from carrying on or prosecuting such work in such manner that another may incorporate for that purpose without competition is not void as against public policy.

* *Proceedings*, Am. Soc. C. E., for October, 1908, p. 1126.

"Contracts in general for total restraint of trade, or contracts for the purpose of creating a monopoly, or compacts having for their object the elevation or depression of the market prices, or to raise or lower the prices of goods and produce, or sales of stocks, grain, and produce on margins, or option contracts whose effect is to corner the markets, are held to be against public policy and void."* Mr. Hazlehurst.

Under the franchise rights, then, it seems that even a limited monopoly by legislative enactment may be voided as against public policy, and this increment of value ceases when the monopolistic right no longer exists, but a monopoly value, predicated upon exclusive possession, is a factor that may be of prime importance and of great intrinsic value.

The business acumen or foresight of an individual or public service corporation in acquiring for use, as a necessary item or element, an advantageous or even exclusive source of water supply, impounding basin, water-shed or reservoir site, constitutes a natural monopoly. It is obvious that such possession would have a value commensurate with its present and prospective use under lawful restrictions.

During the full possession and exercise of its franchise rights to supply water to a community, it would seem clear that where, either through the condemnation right of eminent domain or by arbitrament, a municipality seeks to possess itself of the property enjoyed by an individual or corporation with vested rights, the value of any natural monopoly should be given due weight.

Under such circumstances, the worth of this particular item to the community and its value to the owner should be established by evidence, and, to the writer, it would seem that in equity this worth or value is represented by the capitalized cost of construction to a more remote source of supply or less accessible and more expensive natural location. Perhaps in an unusual case, where such source or location is in fact an actual monopoly, its value might be predicated upon actual and prospective net earnings capitalized, but hardly in perpetuity.

Under such conditions, monopolistic worth to a community is exactly the same as the value of the item to any one else, and the fact of the franchise grant or gift originally being derived from the public is not reason for abatement of specific value in the case of such natural monopoly.

As against these averments, a legal differentiation might apply to a case where the franchise grant had expired by limitation and where a municipality declined to renew the same or to permit a transfer of ownership with extension rights. Such situation would presuppose a desire for municipal ownership, and this public sentiment and attitude would in effect deprive a natural monopoly of its value as such; nevertheless, the acquisition of property under such circumstances

* "Engineering and Architectural Jurisprudence," p. 72.

Mr. Hazlehurst.

would, at least in equity, seem to partake of prescription and confiscation without due recompense, contrary to good morals, if not to sound public policy.

Arbitrary acts and sharp practice should be decried alike in the individual and the community, and public policy as well as a high moral conception should direct opinion so as to prevent an unfair treatment of vested rights in the interest of private or public gain, and especially should this be so where an endeavor is made to destroy values of such quasi-public enterprise as a water-supply plant.

As a specific case, coming within the knowledge of the writer, and illustrating the right of a municipality to secure to its use through condemnation certain items, features, or interests, excerpts are made from the case of Wood *v.* City of Mobile, Circuit Court, S. D., Ala., February 8th, 1900; appealed April 2d, 1901:

"The legislature of the State of Alabama by an act approved Nov. 30th, 1898, authorized the City of Mobile to provide, maintain, and operate systems of water-works and sewerage; and by act approved Nov. 30th, 1898, authorized the City of Mobile to make and issue bonds for building, purchasing or otherwise acquiring systems of water-works, and to secure said bonds; and by an act approved Dec. 10th, 1898, authorized the City of Mobile to condemn by eminent domain the outstanding interest in what is known as the 'Mobile City Water-Works;' under the statutes, the City of Mobile condemned, by exercising the right of eminent domain, and the interest of Walter Wood in what is known as the 'City or Stein Water-Works,' such interest being $54\frac{28}{100}\%$ of the whole. The decree of the Probate Court condemned the lands for public use, and vested the title in the city, all and singular, the perpetual right to all the interest of every kind, character and description belonging to Walter Wood of Philadelphia, Pennsylvania, whether the same be legal or equitable in and to those water-works known as the 'Mobile City Water-Works.'"

Under the theory that the proceedings in the Probate Court were void, the plaintiff instituted in the lower Court an action of ejectment to recover possession. Under this plea, the plaintiff, Wood, contended that:

"The Probate Court had jurisdiction and power to condemn by eminent domain only so much of the land and such interest in the property and lands of the plaintiff as were necessary for the use of the new water-works system, and that only an easement or right-of-way over such lands as it actually needed for the construction of its new water-works system could be condemned by the city, and that, in so far as the decree of the Probate Court went beyond this, it was absolutely void and passed no title to the city; and further, that even if the property was legally condemned, yet if part of it was never used by the city for the public benefit, such part reverted to the original owner, and he could receive it back in ejectment."

Held in Part by the Court.—"As to the contention that part of the property of the old Stein Water-Works has reverted to the original

owner because of non-user after condemnation, it may be admitted that the general rule is that when a corporation, in the exercise of eminent domain, acquires for a public purpose an easement in land, its right and title to the property so acquired are dependent upon the use of the property for public purposes; and, when such public use is abandoned, the right to hold the land ceases, and the property reverts to its original owner. But the plaintiff in error in that case can take nothing under this admitted rule, for two reasons: First, under the statute of Dec. 10th, 1898, more than an easement was condemned; to-wit, all the right, title and interest, legal and equitable, on the basis of which full title it is supposed the owner was fully compensated, and the property thereupon became the absolute property of the corporation; second, because, in the agreed statement of facts in this case, it is expressly admitted that 'the City of Mobile has used, and is still using, the whole of the property and franchises acquired from the plaintiff in error to supply water to the citizens of Mobile, and that the said Mobile City Water-Works are now connected by iron pipes with the new water-works system built by the City of Mobile under the act of the general assembly of Alabama, approved November 30th, 1898.'" Mr. Hazlehurst.

In this cause the plaintiff sought to establish the fact that, only an easement being necessary to the City of Mobile, it could only condemn a right of way over lands in controversy, maintaining that any item or property not actually required for the public use should revert to the original owner. The admissions in the case seem fully to justify the decree.

Considering the converse, where the effort is to force upon a municipality, exercising the right of eminent domain, all and sundry items, some of which are perhaps undesirable or unnecessary to its use, it would seem that in such matter the principle may be applied toward the acquisition of some natural monopoly or coveted feature, neglecting other items whereby their value may be depreciated or destroyed, provided values be considered broadly, the appraisement being based on existing conditions and determined facts, and with due regard to the principle that, after sucking the orange or skimming the cream, values both of the substance taken and remnant left must, in all equity, enter into the final consideration.

Allen Hazen, M. Am. Soc. C. E.—The speaker is very much interested in the subject of water-works valuations, and was fortunate in being able to read a large part of the manuscript for this paper and to talk over various matters with Mr. Metcalf during its preparation. Mr. Hazen.

The decisions of the Maine Courts have certainly thrown a great deal of light upon the methods of valuing water-works properties, and the speaker is extremely glad that these matters have been presented to the Society.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

FOUNDATIONS FOR THE NEW SINGER BUILDING, NEW YORK CITY.

Discussion.*

By Messrs. O. F. Semsch, Eugene W. Stern, and Edwin S. Jarrett.

Mr. Semsch.

O. F. Semsch, Esq.—Mr. Thomson states that some of the columns of the Singer Tower were anchored to the footings in order to provide for the uplift caused by the wind pressure on the building. A brief description of the wind bracing, and more particularly of the method of calculating the load due to wind pressure on the footings, may prove of interest. It was assumed that the tower would be exposed to a wind pressure of 30 lb. per sq. ft. from the top to the roof of the 14-story main building, the lower portion being sheltered from wind on all sides by the surrounding buildings.

The wind moment equals 23.7% of the moment of stability, both moments being taken about the base of the tower. As the Building Code of New York City specifies that the wind moment of any structure shall not exceed 75% of its moment of stability, the design is well within the limits of the law.

There are eleven sets of braces to resist the wind pressure in a northerly and southerly direction, and ten braces in an easterly and westerly direction. These extend up to the thirty-third story; below the fourteenth story there are two additional braces in each direction. Each wind brace may be said to consist of a vertical truss 12 ft. wide

* This discussion (of the paper by T. Kennard Thomson, M. Am. Soc. C. E., printed in *Proceedings* for October, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Semsch.

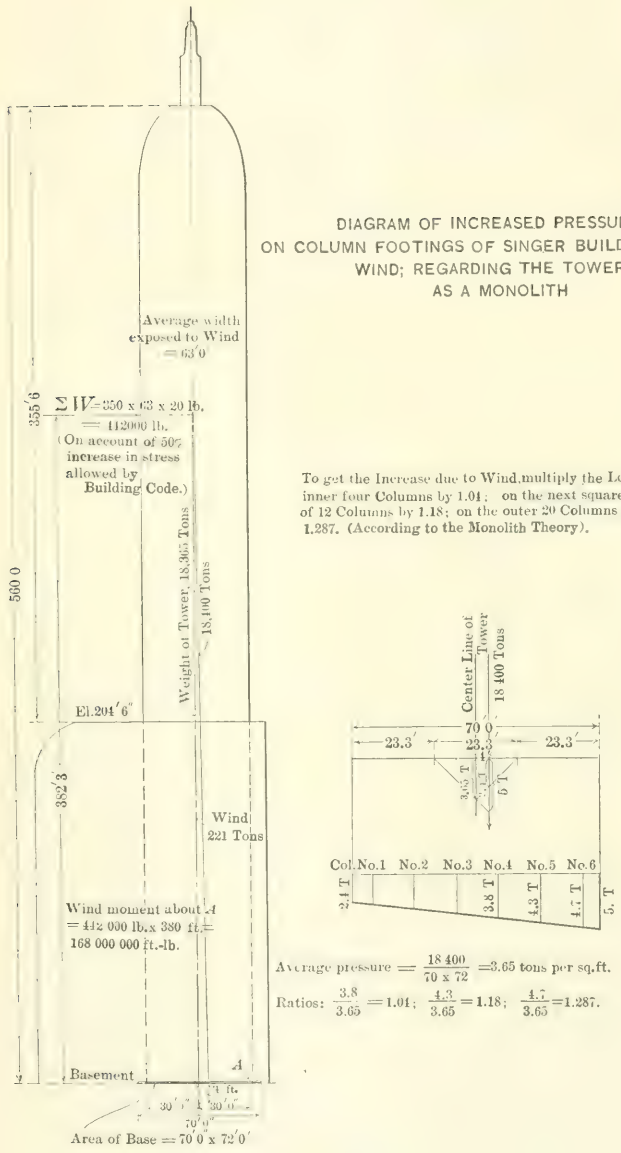
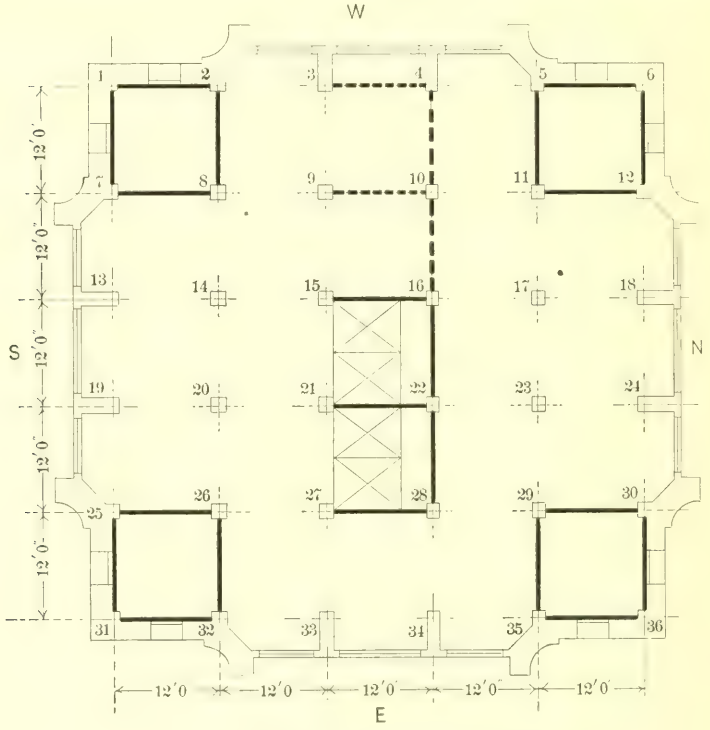


FIG. 12.

Mr. Semsch. and about 500 ft. high, formed by two lines of columns with cross-latticing between them.

On account of the unusual architectural treatment of the tower façades, it was impossible to install a system of bracing extending across the entire width of each side, which, of course, would have been the most natural thing to do. Each front of the tower consists of five equal bays, the three middle ones being combined into what

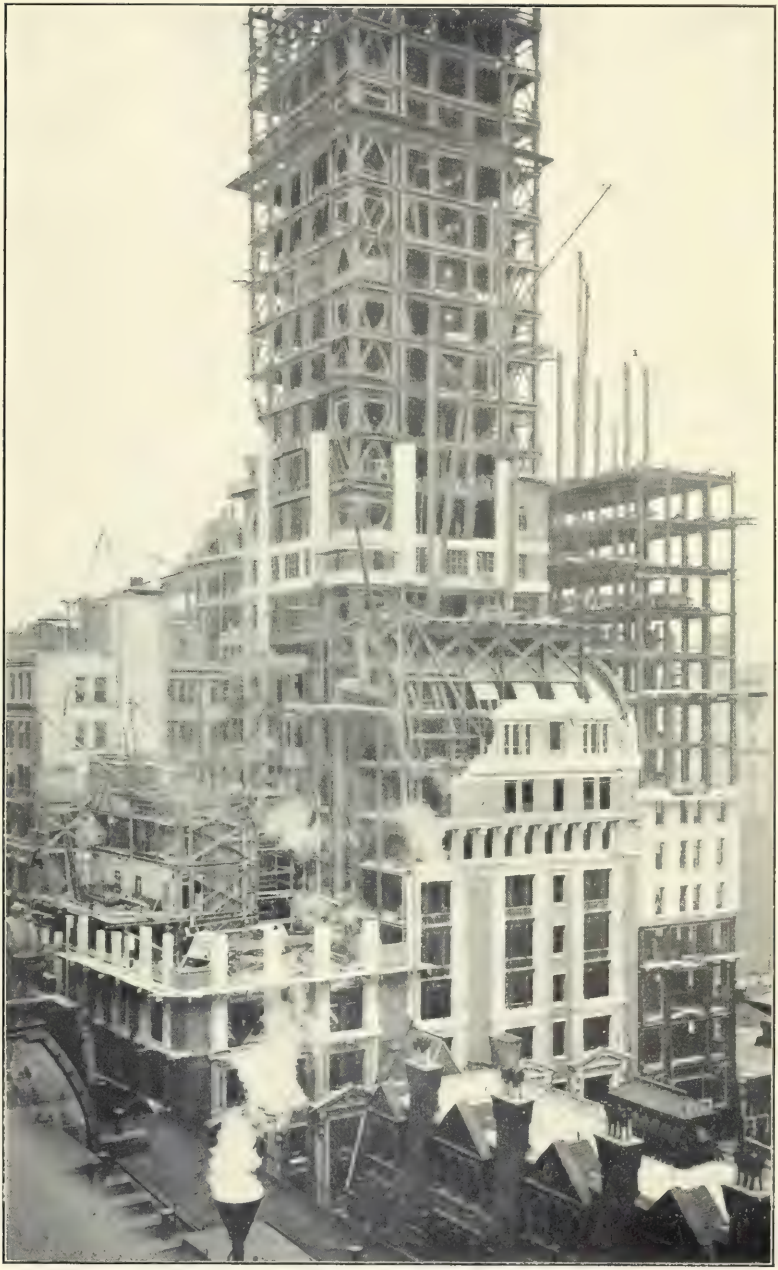


PLAN SHOWING LOCATION OF WIND BRACING IN SINGER TOWER

Fig. 13.

is practically one huge window about 36 ft. wide. This window projects several feet beyond the wall line; it was impossible, therefore, to run any bracing across it. Accordingly, four braces were placed in each corner of the tower, forming really four small towers, each 12 ft. square, and the others were grouped around the elevator shafts in the center of the building.

Each brace was calculated so that it could stand alone and resist its share of the wind pressure. Thus, if ten of these braces were to



THE SINGER BUILDING, NEW YORK CITY, DURING ERECTION.

be set side by side, with a plate, about 60 ft. wide by 500 ft. high, placed against them, they would be capable of resisting safely a pressure of 30 lb. per sq. ft. on that plate. Mr. Semsch.

This method of calculating the wind load, according to what the speaker terms the "brace" system, to distinguish it from another method to be mentioned later, resulted in concentrating that load, as far as the foundations were concerned, under the sixteen columns at the four corners of the tower and under the elevator-shaft columns in the center. The designers, however, were sure that in reality the entire wind load would not be transmitted to the footings by these columns alone, but that some pressure would be carried down by each column of the tower, whether or not it formed part of the bracing. In order to get some idea of what this load for each column might be, the tower was regarded as a monolith, and a calculation was made of the distance the resultant of the wind load and the total weight of the tower would fall outside of the center at its base. This distance was found to be 4 ft. From this was ascertained the ratio according to which the average pressure under each column would be increased on account of this eccentricity. In this way the amount of wind load that each column would transmit to the footings was found, according to what might be termed the "monolith" system.

There were then two sets of calculations, and finally, in accordance with the suggestion of Mr. Ernest Flagg, the architect, the average between the loads obtained by each method was taken as the wind load on the footings, for it seemed reasonable that the correct solution of the problem lay somewhere between the two. In designing the anchors, however, the full uplift developed by the braces regarded as standing alone was taken.

Messrs. Boller and Hodge, who made a separate calculation, assumed that two-thirds of the entire wind load would be transmitted to the footings by the columns forming part of the wind braces, and the remaining third by the columns outside of the wind braces. Wherever the wind load at the base of a column amounted to less than 50% of the combined dead and live loads, they disregarded it altogether—according to the practice followed in bridge designing.

The wind loads given by Mr. Thomson in Table 1 were really reduced by one-third before being applied to the footings, because the Building Code of New York City allows an increase of 50% in stresses when calculating for wind load.

According to the architect's method of calculating, the total load on the tower footings proved to be, in round numbers, 30 000 tons. This, divided by the total caisson area, 2 794 sq. ft., gives an average pressure of 10.7 tons per sq. ft. The load is not evenly distributed, however, but varies from 15 tons per sq. ft. in some places to 5 tons in others. Messrs. Boller and Hodge allowed 15 tons per sq. ft. for

Mr. Semsch. live and dead loads, and 22½ tons per sq. ft. for live, dead, and wind loads. According to either method, the tower is amply safe, and its rigidity and solidity are really remarkable.

As mentioned by Mr. Thomson, several of the caissons were out of plumb a few inches, and one was out about 1 ft. Considering the depth, 90 ft., this was not to be wondered at. However, when the owner heard of it, he accepted the situation as soon as the Foundation Company had agreed to put a 12-in. bed of concrete over the entire area, and around the tops of the caissons, tying them all together.

This precaution was really not necessary, because the surrounding sand and hardpan held the caissons so securely in place that they could not have moved, but the owner very properly took the position that he was entitled to caissons which would be capable of standing up without any earth around them whatever.

This method of solving the difficulty may be used as a precedent by engineers if their clients ever refuse to accept caissons on account of their being out of plumb.

Mr. Stern. EUGENE W. STERN, M. AM. SOC. C. E.—Mr. Thomson has mentioned the inaccuracy of wash borings, and the speaker wishes to endorse this strongly. The results of these borings must be examined very carefully and, if possible, compared with others taken in the neighborhood, and even after this is done the results are not always to be trusted. In one very serious case, wash borings were taken, which showed rock 30 ft. below the surface, and the foundations were designed accordingly. On sinking the foundations it was found that instead of solid rock, the bottom consisted of broken stone filled in to a depth of 20 ft. on poor, soft bottom, the site having been originally a pond which was used as a convenient dumping ground for whatever was excavated in the neighborhood. Had this been known before the foundations were designed, many thousands of dollars would have been saved to the owners.

The method of placing a contract for foundations, namely, a lump sum to a certain depth, and an extra price per cubic yard for everything below that depth, commends itself also as being fair to both owner and contractor.

It would be interesting to know why it was decided to carry the foundation to rock after hardpan had been reached. Was not this considered sufficiently good foundation? The hardpan excavated in the neighborhood of the Singer Building is practically of the consistency of concrete. The speaker has had a specimen exposed in his office for months without the slightest sign of disintegration.

Mention is also made of the care required in sinking caissons in order to avoid tilting. This is, of course, very important and too much care cannot be taken in this matter. It is money well spent. A dollar spent in avoiding misplacements will save hundreds later in trying to rectify them, and, in some cases, correction is impossible.

It is necessary, not only that the work be started exactly right, but also that continual and careful observations be made with surveying instruments, in order that accuracy of position may be maintained. Mr. Stern.

The methods of sinking pneumatic foundations have been carried to a great degree of refinement in New York City, and perhaps nowhere else in any country has so much work of this kind been done, and so much experience been gained. The Moran air-lock is undoubtedly the greatest improvement that has been made in many years in connection with the actual processes in pneumatic work. There are some things, however, which the speaker's experience suggests as being capable of improvement, namely, that working chambers which are usually made of wood might better be made of reinforced concrete; and that the shafts connecting air-lock with working chamber are usually made too small to permit a man to climb the ladder in safety and avoid the hoisting bucket.

The speaker also agrees with Mr. Thomson that the process of sinking should be continuous, and that there should be no stoppages. If this is not done, the danger of the caisson being "hung up" is very great, and it is sometimes impossible to start the sinking again—even by loading it with cast-iron blocks—without blowing out the air; and this should never be done in building construction, as it is very likely to endanger the foundations of adjoining buildings.

Continuous sinking is not possible where the form method of placing concrete is used rather than the coffer-dam method, and, while the latter method is perhaps a little more expensive in first cost, it is very questionable whether it is not cheaper in the end, for it enables the work to be done more accurately and quickly.

Edwin S. Jarrett, Assoc. M. Am. Soc. C. E.—The speaker dis- Mr. Jarrett.
agrees in one particular with Mr. Stern. In so far as he criticises adversely the practice, exemplified in the Singer Building foundation work, of moulding the concrete piers which surmount the caisson inside of temporary forms and of removing the forms before sinking the moulding piers, his views are not tenable. He states, in effect, that the long delays necessary at the various stages of the sinking operations to insure the setting of the concrete before it is safe to remove the forms and to continue the sinking, have most injurious effects. The opportunity thus given for the increase of friction by the settling and compacting of the soil around the piers compels a recourse, as Mr. Stern views it, to excessive blowing in order to get the caissons to resume their downward movement. This blowing, in his opinion, very often throws the caissons out of plumb and in general results in the loss of control of their movements. He favors a reversion to the old style of permanent coffer-dam forms because, in his belief, they allow a continuous sinking and thus insure, with reasonable care, complete control and better average results.

It may be stated at once that the continuous sinking of caissons

Mr. Jarrett. is a partial insurance against unskilful methods and careless handling. On the other hand, with proper care and the skill acquired by long experience, caissons may be held up almost indefinitely and sinking may be resumed at any time without any bad results, it being premised, of course, that all the obvious precautions shall have been taken. The specific evil dwelt upon by Mr. Stern—the sinking of caissons out of plumb—results only from careless work. The presence or absence of a permanent coffer-dam form has little to do with it. If a structure is carried plumb to a depth of, say, 20 ft., it is not easy to throw it out of level. Only such excessive blowing as would allow material to flow in under the cutting edge could accomplish it, and such blowing is reckless and is not allowed by a careful contractor. If the caisson is carrying ample weight, it can be taken down plumb, even after a prolonged stoppage in the sinking. It is concluded, therefore, that the use of removable forms does not necessarily entail the bad work which Mr. Stern considers has resulted from their wide adoption.

Without discussing this particular matter any further, it may be stated that a small economy and an increased efficiency have resulted from this improvement in caisson work for buildings.

The construction and methods described by the author have been brought out somewhat fully in the technical journals. Mr. Thomson, however, has dwelt on the novel feature of this particular work, namely, tunneling through the hardpan from caisson to caisson and underpinning to rock a completed pier more than 50 ft. high. Aside from the interest of this operation, it has a significance which might, under conceivable circumstances, be full of possibilities. The ease with which tunnels, conduits, and the like could be carried through this hardpan as compared with the great difficulty of work, either in the quicksand above or in the rock below, might be of no small consequence. As a rule papers of this kind make no mention of certain emergencies which arise in such operations. Accidents occur in the progress of foundation work, and require great skill and nerve in handling, particularly when the work is being carried on close to heavy buildings. In such locations all the quicksand released will probably come from under the adjoining heavy structures.

It may be pertinent to mention an incident of this kind which occurred in the sinking of these foundations and shows the possibilities of damage, the great care which must be exercised, and the troubles that will arise in spite of that great care.

The steam for operating the compressors on this particular work was supplied by the New York Steam Company, the pipes of which are laid in the streets in the lower part of the city. All steam for such work in Lower New York City is thus supplied, as there is no available space upon which to erect boilers. It is necessary, of course, to keep the compressors in continuous operation, because there are

always two or three caissons on the way down, and the loss of air would cause an inrush of quicksand, which it is absolutely necessary to prevent. To do this, the air pressure must be kept up, and of course the steam must be steadily supplied. Mr. Jarrett.

While this particular work was being done, the New York Steam Company's plant caught fire one night and, as the fire spread, boiler after boiler went out of commission. The air pressure began to decrease. There was one caisson about 15 or 20 ft. down, another more than half way down and a third was close to the hardpan. Had all the pressure been taken from the caissons without making provision to keep out the quicksand, it is extremely likely that great damage would have resulted. Having ascertained that all the steam pressure would be withdrawn, quick action was necessary. The only course open was to flood the caissons. Streams of water were thrown into each, and the working chambers, together with the shafts connecting them with the air-locks, were filled as rapidly as possible to a height above the water-level in the soil. There was thus obtained inside the caisson a compensating pressure against the water in the soil, which eliminated the danger that quicksand might enter.

After the air pressure was taken off, the caissons remained in the condition described, full of water, for from 24 to 36 hours. The compressors were then started, and air was applied to each caisson cautiously, some of the water being blown out and the remainder being forced back into the soil by the air pressure. When the water subsided it was found that in no caisson had quicksand entered the working chamber, but it was in substantially as good condition as when the air pressure was taken off.

The flooding of caissons is resorted to quite often to drown out fires which in timber caissons are of not infrequent occurrence. The methods resorted to in order to meet such emergencies are the natural ones, but very often, in the middle of the night, with no one around to think and act quickly, and with no one on the ground who has been through a similar experience, such situations are hazardous.

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PAPERS AND DISCUSSIONS

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NICKEL STEEL FOR BRIDGES.

Discussion.*

BY MESSRS. CHARLES EVAN FOWLER, M. F. BROWN, H. P. BELL, L. J. LE CONTE, W. K. HATT, JOHN C. OSTRUP, T. CLAXTON FIDLER, ROBERT E. JOHNSTON, AND ALBERT LUCIUS.

Mr. Fowler. CHARLES EVAN FOWLER, M. AM. SOC. C. E. (by letter).—The engineering profession and the metallurgists engaged in the manufacture of structural steel are certainly to be congratulated upon the extensive and careful investigations made by Mr. Waddell into the use of nickel steel for structural purposes, and also upon the work that has been done by F. C. Osborn, M. Am. Soc. C. E., in assisting in this valuable work.

The writer was one of the first to adopt the use of soft medium steel for structural purposes, as covered by "General Specifications for Steel Roofs and Buildings," and is still of the belief that, for a great many years, nothing better will be found for short-span bridges.

The small saving in cost by using either medium steel or nickel steel, would hardly be a valid reason for making a change, and, even while it might be considered advisable by some engineers to use nickel steel for medium-length spans, it will undoubtedly be many years before soft medium and medium steel are entirely displaced in the building of short and medium-length spans.

The great ease with which members can be fabricated from soft

* This discussion (of the paper by J. A. L. Waddell, M. Am. Soc. C. E., printed in *Proceedings* for September, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

steel, and the reliability of the structures with only the usual amount of fair reaming in the shop, are almost unanswerable arguments in favor of continuing the use of this metal for short spans. Mr. Fowler.

It is to be presumed from the data in the paper that the difference in cost of fabrication, as between medium steel fully reamed, and nickel steel, is very small; so that, for spans of considerable length, and for very long spans, there can be little question that if nickel steel proves to be as reliable as the tests stated in the paper would indicate, it will come into extensive use in the near future, more especially as this would enable the engineer to use spans of several hundred feet greater length than is possible at present. The writer, however, has not checked Mr. Waddell's figures, on which the cost of long spans was compared, but it would seem doubtful if it would be possible to make an increase to the extent of 500 ft., mainly on account of so many other factors than the mere cost of the metal entering into the cost of such structures.

The data available for reference by the writer, in addition to this paper, would seem to indicate that the machining of nickel steel would be much more difficult than that of ordinary carbon steel, although, with the small percentages discussed in the paper, for actual use this would not seem to be a very serious matter, and could only be determined definitely by actual experience in the shop.

The tests also seem to indicate that the opinions held in the past as to nickel steel are in the main correct, and that the effect of the nickel is quite uniform. Would it not be well, however, to make a careful investigation of acid nickel steel, as there is no question that for a high grade of steel for long spans, the acid process is somewhat better than the basic.

With reference to the future cost of nickel, it would hardly seem likely that even with the additional deposits which have been discovered, the cost would be very greatly reduced, and it would be well to know whether or not the author has investigated the use of ferruginous nickel for making the alloy, which would be just as effective, and of very much less cost.

It will undoubtedly be necessary to make a great many additional tests before nickel steel can be put into general use, and it is to be hoped that some of the large steel manufacturers will undertake this work, which, to the necessary extent, can hardly be carried on at private expense.

M. F. Brown, M. Am. Soc. C. E. (by letter).—One of the most important facts to be determined in connection with the use of nickel steel for bridges is the proper proportion of nickel and other elements to make the resultant alloy the most adaptable for the purpose intended. Dr. Waddell is evidently of the same opinion, but it seems to the writer that his paper does not give enough prominence to this fact. Mr. Brown.

Mr. Brown. The difficulties in the way of a private individual who may undertake to determine these proportions are apparent; and certainly few, if able, would be willing to devote the time and money necessary to such a solution, as this will hardly be found except after the lapse of years. It is probable that nickel steel, as a material for bridge spans of ordinary length, if it is ever commonly used for such purposes, will have to pass through some such period of development as has structural steel during the last fifteen or twenty years, and be specified with proportions giving elastic limits ranging from those but slightly greater than carbon steel up to that as high as possible, consistent with fabrication under refined shop methods. From these extremes will probably issue a generally accepted material adaptable to ordinary shop methods of manufacture. Whether or not this material will be similar to that proposed by the author is impossible to determine; but it seems to the writer that just because two melts of steel having practically identical proportions give a product of good quality which can be readily fabricated into structural members, it is a little hasty to conclude that "it is not likely that any great improvement in the characteristics of the future plate-and-shape steel, as compared with those of these melts, will be affected."

The author's tests are well chosen to show the suitability of the material for the purpose intended, and certainly indicate the fitness of nickel steel as a material for bridge construction in spans of ordinary length, these being of the greatest interest to most engineers. Long spans, while of great interest in themselves, do not come within the range of experience of the majority of engineers, and can be treated in a measure apart, being of such bulk that special material more particularly adaptable to them may be easily obtained if desired.

The tests for resilience are somewhat surprising, the general impression being that nickel steel has considerably more resilience than ordinary carbon steel; but the difference is not enough to prejudice its use. The corrosion tests are interesting as showing some comparative measure, with steel, of the action of gases, etc. It would have been interesting if these tests could also have been compared on an iron basis.

The writer is not disposed to agree with the author that the results of his investigation make it "practicable to write specifications for nickel-steel bridges which will possess the same strength, rigidity, and general excellence of design as the best carbon-steel bridges that are being built to-day." Anything like a general specification at the present time is attempting too much. The author's tests, valuable as they are in affording an indication of the use of the material, are too few to serve as a basis upon which to write a complete general specification. If, however, as seems probable, the author intends these specifications to serve as a guide only until further information is

available, then they can serve a valuable purpose. As applied to the material as specified, the only criticism the writer offers concerns the compression formulas, which are based upon only six tests of full-sized members. This information appears to be too meager to develop a compression formula which will inspire confidence in the minds of conservative designers, and, until further tests are available, prudence would suggest reducing these units to an undoubtedly safe value. Mr. Brown.

An enormous amount of labor is indicated in the preparation of the diagrams giving weights of bridges and their comparative costs with all carbon steel and mixed nickel and carbon steel. It seems almost a pity that the author should have gone into this so deeply when it is considered that the specifications for the two materials can hardly be expected to maintain the relations existing when the diagrams were prepared. However, they will illustrate their purpose, and the profession is in a position to appreciate them.

These remarks are not intended in a spirit of criticism of this most valuable and timely paper, and the writer wishes to add his word of appreciation of the spirit which makes such a paper possible and available to other and less gifted engineers.

H. P. Bell, M. Am. Soc. C. E. (by letter).—Dr. Waddell's paper is likely to be of great service, not only to the profession, but also to the general public. Mr. Bell.

When it is considered that manufacturers can produce steel wire with an ultimate tensile strength of 100 tons per sq. in., and plate-and-shape steel of about one-third the same tenacity, it is evident that the makers of the latter should be encouraged to improve its quality as much as possible.

It is a common expression that "it is the last straw that breaks the camel's back," and this aphorism comes forcibly home to the engineer who is designing a long-span bridge for modern double-track railway and other kinds of traffic combined. The use of nickel steel, as suggested by Dr. Waddell, may solve some problems not otherwise easy of solution.

As soon as engineers begin to manifest practically a desire to have a superior quality of nickel steel for bridge purposes, the makers themselves will take up the subject in earnest. The demand must come, before the quality of the supply will be as good as it can be made, and it probably will take some time to find out how to manufacture a superior kind of nickel steel for structural purposes. It is not to be supposed that an improvement once begun will be suddenly arrested. Upon the face of the facts already ascertained, there is something puzzling about the making of good nickel steel for bridge purposes—something that requires continued action among the makers—to throw

Mr Bell. light upon apparent inconsistencies, and to clear the way to the perfection that is likely to be attainable with due persistence.

Not many years ago metallurgists found great difficulty in the reduction of refractory ores, but, being persistent in their efforts, they finally succeeded to a very large extent, but this improvement may continue for a long time still.

"Resistance to cracking, a property to which the name of non-fissibility has been given, is shown more remarkably as the percentage of nickel increases. Bars of 27% nickel illustrate this property. A $1\frac{1}{4}$-in. square bar was nicked $\frac{1}{4}$ in. deep and bent double on itself without further fracture than the splintering off, as it were, of the nicked portion. Sudden failure or rupture of this steel would be impossible; it seems to possess the toughness of rawhide with the strength of steel. With this percentage of nickel the steel is practically non-corrodible and non-magnetic. The resistance to cracking shown by the lower percentages of nickel steel is best illustrated in the many trials of nickel-steel armor."*

In a table on the page from which the above quotation is taken, there is the record of two specimens of $3\frac{1}{4}$% nickel steel (bars forged down from a 6-in. ingot to $\frac{3}{8}$ in. diameter, with conical heads for holding), the first with an ultimate tensile strength of 276 800 lb. and the second with 246 595 lb. per sq. in. On looking at the reduction of area and elongation, it is seen that neither rises above 6 per cent.

As these ingots were not rolled but forged down, it looks as if the amount of work put upon them, while increasing the tensile strength, had impaired the reduction and elongation. It is in questions of this kind that manufacturers can be helpful to engineers, and it seems to the writer that engineers in specifying what they require, should not fall below the standards of this paper, but rather, if anything, exceed them, because the latter policy would put the manufacturers on their mettle. If it be assumed, in the first instance, that nothing better can be got than that which has been made already, progress and improvement will be slow. Suppose that a large bridge was to be tendered for and a notice such as the following was given:

Every contractor must furnish with his tender ——— bars of nickel steel of such and such size for testing purposes; and other things being equal, the tender possessing the strongest steel will have the best chance of securing the contract, which will not necessarily be let to the lowest bidder, but the contractor who is awarded the work shall bind himself to manufacture steel of at least equal quality with the samples tested.

If such a notice (or one more carefully drawn) were used, the probability is that a superior class of nickel steel could be had in the shortest time possible.

* See "The Mechanical Engineers' Pocket-Book," by William Kent, p. 408.

Mr. Bell.

It was not to be expected that a preliminary inquiry, however exhaustive, into a subject of such magnitude and importance to the profession and the public, would cover all the ground necessary to be gone over. This is a matter of time and probably also of expenditure, large enough to awaken the sympathy, and engage the active assistance, of all those who may be commercially interested in the making of nickel steel for structural purposes.

Lately the writer became aware of the fact that a three-hinged metal arch of 1 800 ft. span could be built of carbon steel for a metal weight of 35 000 lb. per ft.; no unit stress on main members of more than 15 000 lb. per sq. in., either in tension or compression; and upon other parts none greater than 18 000 lb. per sq. in.; width of the floor, 80 ft.; live load per foot, for double-track railway and other kinds of traffic, 16 000 lb.

If this same structure were to be built of equal strength of nickel steel, it would weigh very approximately three-fourths of the above, or about 26 250 lb., instead of 35 000 lb. per ft.

It will be evident to those who study Dr. Waddell's paper, that engineers could now safely build upon his specifications, and realize an important economy; and that as the improvement in the manufacture of nickel steel continued, a class of material, better than the manufacturers would probably admit of as practicable at the present time, would soon be secured.

The writer believes that Dr. Waddell and his associates have done a most meritorious piece of work at an opportune time. It was needful that someone should take up the subject, and it is doubtful if it could have fallen into hands more capable, generous, and painstaking.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Experiments on a small scale to determine the unit stress per square inch on the raw material of new metals as they come from the mills is certainly highly desirable information to be used subsequently in the manufacture of bridge members; but this, strictly speaking, is not the material with which the bridge engineer has to deal. He is practically compelled to take the finished members as they come to him from the shops ready for erection. This is what he uses, and, therefore, the tests of full-sized members just as they come from the shops give him practical results of the highest importance, the intrinsic value of which cannot be over-estimated. Past experiments generally show "crippling" apparently somewhat inside of the so-called elastic limit. This, it seems, is undoubtedly due to the imperfections of detailing in shopwork which cannot be entirely avoided in any case. Shopwork detailing 20 years ago was very different from that of the present day, and, moreover, even to-day the work turned out under ordinary conditions may be vastly different from that turned out under a rush order. In the latter case the inspectors are often compelled to slur over many things which

Mr. Le Conte. they would not think of passing otherwise, and the result is inferior shopwork, which reduces not only the strength, but also the life, of the whole structure. This is particularly true of built-up members requiring a large amount of riveting.

This being the case, it is more than interesting to study the author's experiments on well-built, full-sized members of nickel steel. These columns he speaks of as properly designed and properly manufactured. The results reported are certainly highly satisfactory, and are to be commended in every respect. These experiments seem to show that the engineer is now called upon to make his unit stresses a function of the shopwork, instead of a function of the so-called elastic limit. In the West, particularly where the matter of railroad transportation of finished members is a very serious item of expense, it often happens that it is much cheaper to order the different parts of bridge members all carefully matched and punched at the eastern shops. Then, after their arrival at their destination, they are assembled and riveted up by the western shops. As a result, there is generally a mixture of good and poor work, and no formula could come anywhere near fitting the facts in such cases.

Inasmuch as these experiments seem to show conclusively that the ultimate stress which finished bridge members will stand, when tested to destruction, is largely, if not entirely, a direct function of the quality of shopwork, it would seem to be entirely reasonable and proper that every important member of a bridge (tension as well as compression—or in some members both) should be tested systematically with the full loads called for in the specifications before they leave the shops. This requirement, although severe, would certainly develop the weak points in shopwork if any existed. Again, on the arrival of the structural iron at its destination, all members should be examined carefully to see that they have not been injured in transit. And finally, during erection, care should be taken that the members are not injured in handling by the erecting parties. It is an old saying, and a good one, that eternal vigilance is the price of good work.

The more the writer thinks over this paper, the more he is impressed with the fact that it is highly inadvisable to lower the standard unit stresses called for in all the standard specifications of the present day, because they are based primarily on long, painstaking, and widespread experience extending over many years of gradual and progressive improvement in shopwork. Furthermore, any attempt at reducing these unit stresses would be practically equivalent to offering a premium on cheap draftsmen and poor shopwork, both of which no engineer of experience in such matters would countenance.

Whenever the quality of shopwork for any reason begins to deteriorate, it is often extremely difficult to find out who is responsible for the unhappy results. Of course, true economy is always in order,

and is certainly entitled to the highest respect, but true economy is also about the rarest article to find on the face of the earth. This arises from the fact that those in supreme charge rarely have any conception of the value of minute technical knowledge and experience in detailing shopwork, and very often discharge a capable and trustworthy man in order to make room for a cheaper substitute. This is too often the case, and, as a result, they never seem to wake up or to realize fully the facts of the situation until some heavy bridge member, being handled by a yard derrick, buckles up right before their eyes. In shopwork, especially, the honest laborer, the world over, is always worthy of his hire. Mr. Le Conte.

This paper is extremely instructive, and shows that the author is a master in the details of his line of business. It is written in a most fascinating way, and the happy results reported are so alluring that one is naturally and unavoidably drawn toward his side of the case. He shows conclusively the great economy in the use of nickel steel for bridge work in general.

The enormous increase in the available length of maximum span—some 500 to 600 ft.—speaks volumes for all future designs in nickel-steel bridgework and structural materials of all kinds. Moreover, nearly 20% reduction in the cost of long spans will certainly put new life into the whole business, and will produce a great and lasting public benefit, the full measure of which cannot be comprehended by the average man.

W. K. Hatt, Assoc. M. Am. Soc. C. E. (by letter).—It may seem anomalous that the nickel steel tested by the writer for Mr. Waddell should show a greater rupture-work (resilience) than the carbon steel, which is more ductile. The values quoted by Mr. Waddell are as given in Table 61:* Mr. Hatt.

TABLE 61.—Tensile Impact Tests.

	Carbon.				Nickel (3.5%).			
	Number tested.	Maximum.	Minimum.	Average.	Number tested.	Maximum.	Minimum.	Average.
Elongation.....	5	32.0	31.0	31.5	4	19.00	13.00	16.5
Rupture-work.....	5	1910	1540	1736	3	2300	1960	2198

The writer has consulted a paper of his, entitled "Tensile Impact Tests of Metals,"† and finds other comparative tensile impact tests of nickel and common soft machine steel, as shown in Table 62:

*The writer has remeasured the drum records for these impact tests, and finds a correction to be made in the values of the rupture-work in the case of the nickel steel. The correct values are those above.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. IV, 1904.

Mr. Hatt.

TABLE 62.

	Machine Steel.			3.15% Nickel Steel.			
	Number tested.	Maximum.	Minimum.	Average.	Number tested.	Maximum.	Minimum.	Average.
Elongation in 8 in.	13	31.6	23.10	27.00	8	29.40	20.70	24.00
Rupture-work.....	..	1 460	1 208	1 558	..	2 216	1 203	1 821

These tests, made in 1900, indicate the same phenomena as those made in 1907. In this same paper are tests on three grades of steel castings (coupons from locomotive driving-wheel centers). See Table 64. These three tests would serve to indicate that the harder metal, with less elongation, may take a higher drop of a hammer for rupture than a softer metal of greater ductility.

This is not always the case, however, as may be seen from Table 63, which is arranged from the publication referred to.

TABLE 63.—Impact Rupture-Work of Various Materials.
Tension Impact with Single Blow.

Material.	Diameter, in inches.	Gauge length, in inches.	Impact. Elongation. Percentage.	Impact. Rupture-work, in foot-pounds per cubic inch.	Static. Tensile strength, in pounds per square inch.
Soft steel.....	0.50	8	27.00	1 358	68 000
Boiler plate.....	1 by 0.5 (rectangle)	8	34.40	1 855	60 000
Soft steel castings.....	0.50	2	33.00	2 315	62 000
Nickel steel	0.50	8	24.00	1 821	85 000
Steel wire.....	0.16	108	0.70	186	115 000
Steel wire annealed.....	0.16	108	9.80	772	83 000
Steel wire annealed.....	0.30	108	13.60	765	71 800
Steel wire.....	0.26	108	5.10	556	109 000

The elongation quoted is that remaining in the broken test bars after the impact test. At the exact time of rupture the elongation is undoubtedly greater by the amount of the subsequent elastic recoil, which latter is no doubt a function of the hardness of the steel.

TABLE 64.—Tension Tests in Impact on Steel Castings.

Character of fracture.	Kind of test.	Elongation.	Rupture-work.	Contraction.
Silky.....	Impact.	30.0	2 160	40.0
Flaky.....	Impact.	19.0	900	18.8
Fine bright.....	Impact.	22.6	2 811	23.2

JOHN C. OSTRUP, M. AM. SOC. C. E. (by letter).—This paper is certainly very exhaustive in its scope, and it is no wonder that so much time was consumed in its preparation, and in making the numerous tests and calculations involved. The information and deductions resulting therefrom are well nigh incalculable to the profession in general and to the bridge engineer in particular. Mr. Ostrup.

Without any outside assistance, it is manifestly impossible for many individual engineers to carry on costly experiments on such a broad scale, and it is more than doubtful whether any steel manufacturer would care to do so.

It is also improbable that the steel producer will look upon the author's conclusions with a kindly eye; that is, commercially speaking, it is in the interest of the manufacturers to oppose any innovation which will require a refitting, either partially or totally, of his furnaces, his shops, tools, etc., without the full assurance of a commensurate increase, and consequent profit, in the use of the new alloy. This again is doubtful, for some time at least, for the author has shown that superiority in economy does not become a decided factor except for the longer spans, or when the price of carbon steel is high.

However, the work has been completed, the profession has received the benefit, and the writer thinks that too much commendation cannot be extended to the author or to his associates in this undertaking.

Taking up various points in the paper, the first refers to the impact tests made to determine resilience.

The author here correctly states, in regard to this method: "* * * the total amount of abuse given to the metal to be a measure of its toughness and not of its resilience."

This is quite true, inasmuch as an impact test, no matter how made, can only give comparative ideas as to the toughness and ductility of various metals, and not their exact resilience.

The resilience of a structural material is determined otherwise, viz., by its ability to absorb potential energy, under a load or stress, and to restore the same when the cause has been removed. In other words, below the elastic limit, resilience is a measure of ability in materials to perform work.

When such load or stress is being gradually applied, the resilience, per unit volume, is expressed as follows:

$$\left\{\begin{matrix}\text{Direct tension, or}\\ \text{direct compression}\end{matrix}\right\} \qquad \text{Resilience} \ldots = \frac{S^2}{2E}.$$

For a beam of rectangular cross-section, such as was used in the experiments, this formula reduces to:

$$\text{(Bending)} \qquad \text{Resilience} \ldots = \frac{S^2}{18E}$$

Mr. Ostrup. where S = Stress per square inch, at the elastic limit, uniformly distributed over the entire area;

S_1 = Stress per square inch, at the elastic limit, in the extreme fibres;

E = Modulus of elasticity.

Neither S nor S_1 can be ascertained with any degree of accuracy, except where a quiescent load is being gradually applied.

In a drop test, assume a weight, P, falling through a height, h; then the total work $= Ph$. Part of this energy is absorbed by the supports, part by the polar moment of inertia of the beam, and the remainder goes to perform potential work. How great that remainder is, it is not possible to determine with accuracy.

On the basis of the above equations, and using values specified in "De Pontibus," and also in this paper, a better comparison is obtained between the elastic resiliences of carbon steel and nickel steel, viz.,

$$\text{High carbon steel} = \frac{40\,000^2}{2 \times 29\,000\,000} = 27.66 \text{ in-lb. per cu. in.}$$

$$\text{Low nickel steel} = \frac{60\,000^2}{2 \times 30\,000\,000} = 60.00 \text{ in-lb. per cu. in.}$$

$$\text{High nickel steel} = \frac{65\,000^2}{2 \times 30\,000\,000} = 70.42 \text{ in-lb. per cu. in.}$$

Or, calling the resilience for carbon steel 100%, we have:

High carbon steel	100%
Low nickel steel	216%
High nickel steel	254%

These percentages would indicate correctly the comparative resiliences were it not for the facts, as mentioned hereafter, which somewhat modify the same.

When thus modified, it will be found that the theoretical comparisons will approximate closely to those found in actual tests by Professor Hatt, as referred to by the author.

Is it not quite true that the above comparative values, or in fact any exact comparison, will only obtain where perfect specimens—*i. e.*, specimens uniform in cross-section throughout—are being tested. By the introduction into the tests of deformed or notched bars an element causing great uncertainty has been added.

Compare, for instance, the flexural resistance of two bars, one of which is of somewhat larger cross-section than the other, but has been notched so that its section modulus (but not its moment of inertia) through the notched portion equals that of the smaller and uniform bar.

According to theory, the two bars ought to carry the same load, but this is not so. The effect of the notchings, by contracting the area of the cross-section, is to set up severe secondary stresses, their magni-

tude being dependent on the relative size and shape of the notches with reference to the un-notched cross-section. Mr. Ostrup.

It has been the writer's experience that these secondary stresses reduce the ordinary strength at the elastic limit by from 20 to 40%, and at the ultimate limit slightly more.

While this gives an idea as to the reduction in the static strength of a notched beam, it shows by no means the reduction in its total resilience. This reduction follows an entirely different law, and the effect of this law is to concentrate a large part of the entire work (done upon the beam) on the weak section.

This law is expressed as follows, when considering an elementary area:

$$d\,w = \frac{M\,d\,\beta}{2}$$

where w = Internal work done,
M = Bending moment,
β = Angle after bending between planes which were parallel before bending.

To those who have broken notched specimens, whether in the testing machine or across their knees, it is a matter of knowledge that the angle, β, will be far greater at the point where the cross-section is reduced, hence the work done at that point will increase and the total resilience of the section decrease correspondingly.

That the author was aware of these facts, he indicated by finally intending to use plain bars, and it is the writer's opinion that all tests should be made on perfect specimens and, furthermore, that the "impact" test of any kind of steel be entirely dispensed with.

The results obtained by the corrosion tests, and the lesson these contain, are certainly very instructive, if not somewhat ludicrous. Thus, for instance, in the acid test the author's high nickel steel lost 94% in 160 days, whereas that tested by The Osborn Engineering Company lost apparently in the same space of time only about 2 per cent.

This, however, is not mentioned as a reflection upon the tests themselves, but rather upon the fact that, so far as the writer knows, the profession has no standard specifications covering the method of performing either corrosion tests, or many other equally important tests, upon structural materials.

It has been the writer's experience, in this matter, that nearly every testing bureau or testing laboratory uses a method of its own.

In his specifications for nickel steel, the author advocates its use for the floor systems of bridges, saying "* * * the floor system shall preferably be of nickel steel." To this the writer agrees heartily, inasmuch as there should result a considerable saving in the web by its use.

Mr. Ostrup. The economical depth of a stringer, or any plate girder, being dependent upon the allowable unit stress in the flanges, a much shallower depth with nickel steel could be used. Then again the deeper the web (where no intermediate stiffeners are used) the thicker the web must be, in order to satisfy the requirement for a reasonable ratio between unsupported depth and thickness.

In other words, with an allowable ratio of 1:60, a web plate 40 in. deep, using 6 by 6-in. angles, would require a thickness of $\frac{1}{2}$ in., whereas with nickel steel, for the same case, the depth of the web would only be 31 in., and its consequent thickness $\frac{3}{8}$ in. Since there is nearly always a great surplus of metal in the web, the waste would be less when using nickel steel.

In his comparisons of costs between carbon steel and nickel steel, with reference to plate girders, it would be interesting to know whether or not the author actually used the reduced economical depths allowed by the use of the stronger alloy.

In many standard specifications giving working stresses for members under a direct stress, no mention or provision is made for stresses due to bending from their own weight, or for other secondary causes. This omission is more general where tension members are concerned. That such bending stresses are by no means negligible can be readily seen by reference to a recent case, where the sectional area of some members ranged between 700 and 800 sq. in., and the lengths between 50 and 60 ft.

In "De Pontibus" the author provides for such bending stresses, and whenever done, the use of nickel steel again will show an advantage, inasmuch as these stresses will be considerably reduced.

The author does not specifically mention suspension bridges, where he demonstrates the economy in the use of nickel steel and its superiority for long spans, but such bridges should undoubtedly be included. The case is nearly parallel to that of cantilever bridges, since its dead load is great compared to the carrying capacity, and any reduction in the dead load is also here of the greatest importance.

Furthermore, the use of nickel steel in tall office buildings should be considered in the near future. Some of these buildings in New York City contain from 25 000 to 30 000 tons of steelwork, and, whereas the plate girders or beams might show little or no economy in the use of nickel steel, the writer thinks it highly probable that the columns would show considerable economy in weight and cost, and also advantages in a reduction of the space occupied.

The author states as his opinion that, when used in large quantities, the price of nickel would probably be reduced from 30 cents to 25 cents per pound. Is not that contrary to the laws of supply and demand? Would not the use of nickel steel, or nickel, in large quantities equally well cause an advance in its price?

Considering in its entirety the whole question of the adoption for bridges—or more broadly speaking for structural purposes—of nickel steel, the writer cannot add his voice too strongly to those who are in favor of an alloy which shows so many patent advantages. Mr. Ostrup.

There will undoubtedly be some who, for one reason or another, or for no reason at all, will protest against its introduction. Many of our professional forefathers did the same when Bessemer steel was first spoken of, claiming that they had no confidence in its strength or suitability for structural purposes.

T. Claxton Fidler, Esq., (by letter).—This paper conveys information which is of much value and interest, and it goes so thoroughly into the economic advantages of the new material that very little can be said on that head. Mr. Fidler.

The calculations of steel quantities for various spans are very complete, and, although the methods of calculation are not given, the writer cannot doubt but that the special conditions which must affect the design of all compression members, and also those which arise from the undetermined value of the dead load in long spans, have been taken into account. It is only on these points that he ventures to add a few words.

The experiments seem to justify a safe working stress, f_n, for nickel steel, which is nearly $\frac{5}{3}$ the working stress, f_c, that would be adopted for carbon steel; and this ratio of 5 to 3 seems to be applied to eye-bars and also to columns of the ordinary proportions of length, l, to radius of gyration, r. It follows, apparently, that, with the substitution of nickel steel for carbon steel, the sectional area of an eye-bar might be reduced to about $\frac{3}{5}$, making $A_n = \frac{3}{5} A_c$. In the case of a column carrying any given load, however, it would not be found practical to effect the same reduction, because it would be difficult to preserve, in both materials, the same ratio of l to r.

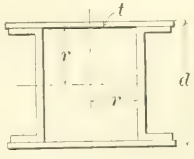

Fig. 76.

Let us suppose, for example, that, in a bridge of given outline and dimensions, we have to design a column of given length, l, to carry a given load; and, first, using carbon steel, we have fixed upon the best diameter, d, radius of gyration, r, and thickness of plate, t, giving a sectional area, A_c, proportional to the product $r\ t$. Then if we proceed to substitute nickel steel for carbon steel and keep to the same radius, r, we should have to reduce the thickness to $t_n = \frac{3}{5} t_c$, and the column would fail by secondary buckling or a crumpling of the thin plate which was already thin enough.

Mr Fidler. On the other hand, if we keep to the same ratio of r to t, in order to avoid this weakness, we should have to reduce the radius to something like $r_n = r_c\sqrt{0.60} = 0.8\ r_c$, or thereabout, and, when the best dimensions are found, the ratio $\frac{l}{r}$ will be greater than before, and the area A_n greater than $0.6\ A_c$.

So far as the compression members are concerned, the economic advantages of the stronger material are only fully realized when the load is great in comparison with the length, l, or, in other words, when there is no liability to buckling; and the advantage is gradually lost as the load becomes less in comparison with the length of the strut. In bridges of the longest span the columns will be of great length, and will be specially liable to buckling; and here we should expect to find, at the same time, that the dead load would be far less in a bridge of nickel steel than in one of carbon steel. The economic advantage of the stronger material, as applied to these long and slender struts, would be comparatively small, just as it has been in the substitution of carbon steel for wrought iron; and, though it could not disappear altogether, it would be less than the advantage of 3 to 5 in the weight of steel required.

On the other hand there is no such limitation in respect to the tension members, and if eye-bars of nickel steel are found to fulfill all the practical requirements, there seems to be no reason why they should not realize the economic advantage to the full. In suspension bridges we should get this advantage throughout the main superstructure; in girders or cantilevers we should get it in the tension members, which form one-half of the structure, while in arches the advantage would be still less.

The writer has long believed that a suspension bridge of rigid construction would efficiently fulfill all conditions of railway traffic, while realizing other important advantages (a view which was supported on one occasion by Lord Kelvin), and, for spans of 1 500 ft. and upward, it would possess (in many locations) a distinct advantage over the cantilever in point of economy as well as in its rigidity under traffic, its unfailing stability, and its safety under wind pressure.

In any case it is evident that the economic advantage of nickel steel, whatever it may be for small spans, will be enormously greater for bridges of very long span where the weight of the steel itself forms the chief part of the load to be carried. The curves of the author's diagrams show very clearly how the weight of steel, w, per linear foot increases rapidly with the span, L, rising toward infinity as L approaches the "limiting span," $\mathcal{L}$, for any given type of structure.

In a discussion of this matter in "Bridge Construction," the writer has given the formula for weight of main superstructure per foot,

$$w = \frac{L\,p\,\Sigma\,\gamma\,\mu' + L\,q\,\Sigma\,\gamma\,\mu''}{1 - L\,\Sigma\,\gamma\,\mu}$$

Mr. Fidler.

while the total weight of the structure is $w + p$.

Here the wind metal is reckoned separately and included as part of the platform load, p, and therefore part of the load carried by the main superstructure.

For the main superstructure, the square inches of steel section required in each member are separately reckoned for the three functions which have to be fulfilled, *viz.*,

For carrying the platform load the quantity ..... $L\,p\,\Sigma\,\gamma\,\mu'$
For carrying the live load and its impact ..... $L\,q\,\Sigma\,\gamma\,\mu''$
For carrying its own weight ..... $L\,w\,\Sigma\,\gamma\,\mu$

The formula is applicable to independent girders, arches, or suspension bridges, but, as it stands, not to cantilever bridges; and for a given type of design the quantities, μ, μ', and μ'', are independent of the material used. The limiting span, $\mathcal{L}$, is obviously reached when $1 - L\,\Sigma\,\gamma\,\mu = 0$, or when $L\,\Sigma\,\gamma\,\mu = 1$, so that $\mathcal{L}$ is inversely proportional to the specific weight, γ, per ton of stress, or directly proportional to the working stress, f. For each type of structure in nickel steel, $\mathcal{L}_n = \mathcal{L}_c \frac{f_n}{f_c}$; and for the suspension bridge we should expect $\mathcal{L}_n = \frac{5}{3}\,\mathcal{L}_c$.

The curve representing the relation of w to L is easily traced; and its general form agrees with the curves of the author's diagrams, as shown in Fig. 77.

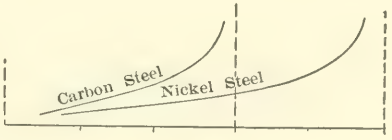

FIG. 77.

ROBERT E. JOHNSTON, ESQ. (by letter).—The writer congratulates the author on the results he has obtained in his investigation of the physical characteristics of nickel steel as compared with carbon steel. Mr. Johnston.

After a careful perusal of the paper, the writer is of the opinion that, with the proportions of nickel determined by the experiments, nickel steel has been proved to be a material which can be safely relied on in the construction of bridges of large span, and that considerable economy can be obtained by its adoption as compared with carbon steel; but it is a question whether the difference in cost will justify its adoption in bridges of small span, because, for structural

Mr. Johnston. reasons, full advantage cannot be taken of the difference in the strength of the two materials, for instance, in the webs of plate girders.

The difference in the strength of nickel steel for rivets as compared with carbon steel, requires that, for the latter material, additional metal must be provided in order to compensate for the area removed from the plate, owing to the increased diameter to be given to the nickel-steel rivets, as stated in the paper.*

Another great advantage of nickel steel over carbon steel is that the former is affected to a less degree by the atmosphere and the fumes from locomotives. This is of great importance in the maintenance of large bridges, for it necessitates a considerable outlay in scaffolding for painting purposes, but, to a certain extent, this is discounted when the two metals are used in the same structure.

The photographs of the columns tested (Plate CVI) would appear to indicate that the unsupported wing of the angles should be made thicker than the other wing, which would have the effect of increasing the radius of gyration, and thus add to the strength of the column. It would add to the interest of the experiments if they included cylindrical columns, in order to determine their strength and also to ascertain the ratio of the thickness to the diameter.

Mr. Lucius. ALBERT LUCIUS, M. AM. SOC. C. E. (by letter).—The writer would most assuredly build long-span bridges of nickel steel. The disclosures in reference to the Quebec Bridge and the Blackwells Island Bridge would not leave a moment's hesitation in his mind. With equal assurance, he would build short-span bridges of soft steel, and would not even be influenced by the possibility that nickel steel bridges might be as cheap or cheaper, simply on grounds of greater perfection. The writer is not sure where he would draw the line, but, in a general way, he would draw it when dead-load stresses exceeded live-load stresses, and would then commence to study the matter in detail.

Dr. Waddell's paper is a complete enough proof that nickel steel is fully available. The writer would prefer heavy bridges of soft steel to light bridges of nickel steel for railroad purposes, because he would consider them steadier under heavy loads and high speeds. When the practical limitations of manufacturing bridges are reached he would use nickel steel to help out, and would probably make the entire construction of nickel steel, and not only a few members. The writer considers this paper to be highly valuable, and believes that the thanks of the Profession are due to the author.

* Page 741.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES HAROLD TUTTON, M. Am. Soc. C. E.*

DIED JUNE 19TH, 1908.

Charles Harold Tutton, the eldest son of George and Sarah Henry Tutton, was born in Tunkhannock, Pennsylvania, on May 10th, 1851. When eleven years of age, he entered a country printing-office, where he remained until he was fifteen. In March, 1867, Mr. Tutton entered the service of the Lehigh Valley Railroad as an engineer, and from 1869 to 1872 attended the Rensselaer Polytechnic Institute at Troy, New York.

Between 1871 and 1873, he located and built the Montrose Railway, in Pennsylvania, and also worked on railroads in New Jersey. From 1874 to 1877, he was engaged in mining and general engineering with I. A. Sterns at Wilkes-Barre, Pennsylvania. In 1878 he was employed in bridge engineering at South Bend, Indiana, but, in 1879, he returned to Wilkes-Barre, and was engaged in bridge engineering and mining until 1882. At that time he moved to Buffalo, where he resided until his death.

Mr. Tutton was connected with the Lehigh Valley Railroad from 1882 to 1888. He designed and built all the Tifft farm improvements, canals, coal trestles, and the shore protection along Lake Erie on the Hamburg Turnpike. He also built that company's coal trestles in Chicago.

From 1889 to 1892, Mr. Tutton was engaged on general work for contractors, and, during this time, built a number of trestles on the Adirondack and St. Lawrence Railroad, the Lehigh Valley coal trestle at Cheektowaga (the largest in the world), the new Buffalo city reservoir, etc.

Since 1893 Mr. Tutton had been connected with the Bureau of Engineering (Department of Public Works), of Buffalo, New York, and had charge of the erection of the steel liberty pole on the Terrace, city dredging operations, Buffalo river improvements, designing and constructing the sewer system of South Buffalo, and constructing the bridge over Cazenovia Creek, city survey, plans for the proposed new water-works, etc. In 1889 he was a candidate for City Engineer, but was defeated.

*Memoir prepared by the Secretary from information furnished by Mr. Tutton's family.

In December, 1882, Mr. Tutton married Fannie C. Draper at Tamaqua, Pennsylvania, who, with three daughters and one son, survives him.

Mr. Tutton was an able and successful engineer, and was known throughout the country as a genius in mathematics. He contributed many articles and papers to engineering magazines, among them, "Flow of Water Through Pipes" in 1896. His personal mathematical manuscripts are marvels to all who knew him.

Mr. Tutton was elected a Member of the American Society of Civil Engineers on October 2d, 1901. He was also a Member of the Western Society of Engineers since 1877; of the Engineers' Society of Western New York from its inception; and of the American Association for the Advancement of Science.

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